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V. 57

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"The Tensile Strength and Diagonal Tension Resistance of Structural Lightweight Concrete," by J. W. HANSON describes the tests employed and results obtained in an extension of a previous study. This phase of the program involves lightweight concrete beams of longer span and lower steel percentages.

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ARTHUR R. ANDERSON and A. T. WAIDELICH open the July issue with a description of the salient design and construction features of a "Precast and Prestressed Concrete Building at the Developmental Center of the Boeing Airplane Company, Seattle." The two-story building, with more than 500,000 sq ft of floor area, was cast, erected, and completed in about 14 months.

•

Results of investigation on the "Properties of an Expansive Cement for Chemical Prestressing" are reported by ALEXANDER KLEIN, TSEVI KARBY, and MILOS POLIVKA. It was established that it is possible to produce concretes having desired predetermined characteristics within a practical range.

•

To increase confidence in the use of the beam method for the analysis of interior cylindrical shells and also provide data for those cases in which the beam method is not applicable, A. L. PARME and H. W. CONNER present "Design Constants for Interior Cylindrical Concrete Shells."

On the Cover—The Huron Towers apartments, Ann Arbor, Mich. The apartments were built using the lift slab method. A detailed description is given starting on p. 1237.

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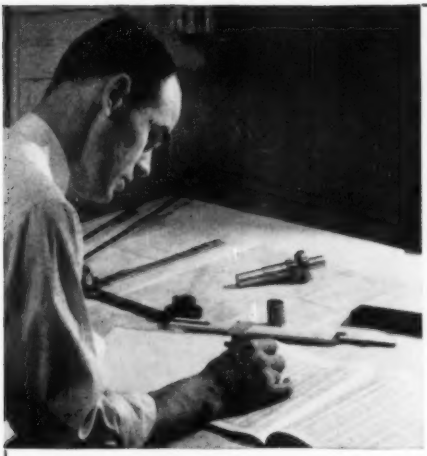

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Title No. 57-67

"Lifting" Huron Towers

By PHILIP N. YOUTZ

Describes the architectural design and construction of the 12-story Huron Towers in Ann Arbor, Mich. This two-tower apartment building was designed for lift slab construction. Foundation design, slab design, and lift operations are described along with the architectural features.

■ ON THE HIGH NORTH BANK of the Huron River near the Veteran's Hospital in Ann Arbor, Mich., rise two towers that reflect new trends in living, in architecture, in engineering, in construction, and in finance. These present the image of the dwellings of the future. Each tower is 15 stories high and the two together provide 360 apartments that have a sweeping view of the University of Michigan in the background and the entire Huron River Valley in the foreground.

REVOLUTION IN LIVING

Huron Towers introduce a new concept of homes that is revolutionary in this area. For the most part Ann Arbor people live in single family houses, the old ones of two or three stories, the new ones generally of one story. The city likes to recall the days not so far distant past when it was a picturesque village inhabited mostly by frugal German settlers. Main Street has a few multiple story buildings but to quote one of the city fathers, "not much goes on above the first floor." Multiple dwellings are modest affairs of two stories and a dozen or more apartments. Homes are usually constructed of wood though the recent ones are brick veneer on wood frames, so they need the protection of an alert fire department. The University of Michigan has followed the village tradition and most of its buildings aside from those of the Medical Center, certain recitation halls, and its recent dormitories, have been kept low in spite of the high land values.

ARCHITECTURE

Architects and engineers value concrete because its plastic phase enables it to conform to almost any visual or structural design. Corbusier and Candela both have illustrated the virtuosity of concrete in their architectural designs. Corbusier delights in demonstrating the massive strength of concrete as in the dynamic structures which support his

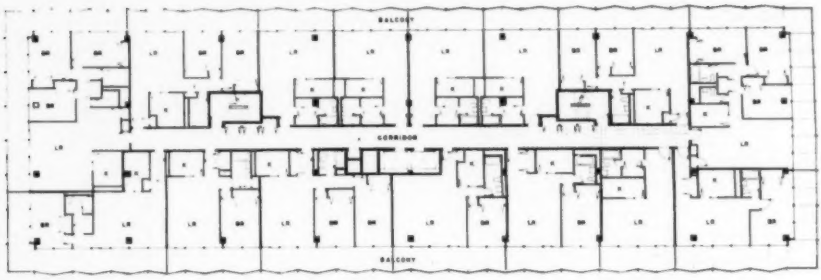

Fig. 1—Typical floor plan

apartment building near Marseilles, France. Candela takes pleasure in proving the lightness of concrete and designs daring churches composed of thin shells not over 2 in. thick. Torroja and Nervi have proved that concrete can take the stresses of the most radical engineering design. Torroja exploits the possibilities of cantilevered structures. Nervi produces vast domes and vaults of precast concrete that appear to be supported on air.

The architects chose concrete for the Huron Towers, because of all manufactured materials it leaves the architect most freedom in designing his building. It imposes no predetermined form and is versatile enough to serve almost any structural requirement. Possibly other considerations were the shortage of steel when plans were started, and the dividend of a whole extra story without added height with which flat slabs reward a thrifty designer. It may have been that they wanted a soundproof structure that would assure the tenants of real privacy. Or the architects may have decided on cantilevered floor slabs so as to provide every family with a maximum balcony for outdoor living.

For the Huron Towers the architects gave strong emphasis to the multiple floors. This expression of the horizontal lines instead of the vertical is logical because the columns are set well back from the edge of the building. The features of the architects' design are the balconies with their textured panels of thin precast concrete serving as parapets and the glass and metal curtain walls set well back from the building line. The edges of floor slabs are cast in a chevron pattern so that the vertical parapet panels attached to them are placed at an angle to the facade of the towers giving the surface of the structures interesting modeling. The effect in the finished building is to assure a varied play of light and color in pleasant contrast with the simple geometric lines of the towers. Glass was used to remove the barrier between indoors and outdoors and to assure the apartments of openness. Balconies and panel parapets give the occupants of the towers a feeling of security. A typical floor plan is shown in Fig. 1.

Philip N. Youtz, Dean, College of Architecture and Design, University of Michigan, Ann Arbor, is especially well qualified to write on the construction of the Huron Towers. He is the developer of the lift slab system used in the construction of these apartments, which were built in his own "backyard."

The Huron Towers form a not unpleasing group with the Veterans' Hospital which formerly stood out starkly against the gently rolling landscape. Before the Huron Towers were built one wondered whether anything less than a grove of giant redwoods perhaps supplemented by a paddock of giraffes would succeed in putting the building in its place visually. The 15 stories of the Huron Towers accomplish this task of essential architectural composition successfully. The three high-rise buildings do not compete with each other but supplement one another so as to achieve an agreeable grouping.

ENGINEERING

The site north of the Huron River on Fuller Street is a bank which rises about 50 ft in 500. Shallow borings indicated well compacted sand and gravel for most of the foundation area. However, to avoid a layer of weak clay near the river, one building was moved 50 ft so that all the footings would rest on reliable ground. An overburden consisting mostly of a peat was removed and at a depth of 15 ft soil was found which could carry a load of 3.5 tons per sq ft.

Footings were designed to run transversely of the towers so as to support four columns each. They were 56 ft long and of the continuous inverted T type, 12 x 3 ft, with a 6 x 5 ft stem. These were placed 25 ft apart on center and each tower rested on nine such footings. This provided for column spacing of 18 x 25 ft and for 36 columns per building. The footings required the use of 3000-psi concrete.

Each building is 70 x 215 ft totaling 15,050 sq ft in ground area. The structures consist of three basements, and 12 stories. The "ground" floor and the upper basement floor have joist type slabs, 14 in. stem + 2½ in. slab, supported by wide flat beams of the same depth. The slab above the ground floor is 10 in. thick to take care of the increased moments due to the 16 ft 8 in. first story height. The upper 12 slabs are 9 in. thick since they carry lighter loads. The three lower floors have added depth to accommodate the heavier ground floor live loads and the heavy basement storage and mechanical equipment loads. All moments and shears for beams and slabs were obtained by standard elastic analysis methods and the sections were dimensioned and reinforced in accordance with Section A600 of the ACI Building Code.*

The specified concrete strength was 4000 psi at 28 days. Excellent strength and workability were attained using a 5-sack mix with 1450 lb

*ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-56)," ACI JOURNAL, V. 27, No. 9, May 1956 (*Proceedings* V. 52), pp. 913-986.

of 2 NS fines and 2000 lb of 10B aggregate (heavy media separation). Air was controlled to between 3 and 4 percent and the concrete was placed by pumping from a central source. The maximum distance for pumping was about 450 ft and the method worked well since the sidehill site permitted down hill flow.

The minimum yield point for steel was 50,000 psi but a lower load was used in the design to minimize deflections. The live load used for the apartment floors was 60 lb per sq ft including the partitions. The live load used for the ground floor and supported basement floors was 125 lb per sq ft plus partitions.

All reinforcing steel for the floors was designed and detailed as straight bars except for a few pieces where anchorage was required for bond. A minimum number of bar sizes was used by varying the spacing and the length requirements of flat slab construction, and by adopting staggered patterns.

Two stairwells, a tower for elevators, a duct shaft, a boiler, and an incinerator stack, were designed in concrete and cast-in-place after the slabs were in position. Intermediate steel dowels were extended through the edge forms and bent down into the walls for these shafts which varied in thickness from 6 in. in the upper stories to 10 in. in the lower stories. Extra, heavy bars were placed at the corners of the shafts to resist the shear from wind loads and all walls thus became bearing walls under finished conditions. Slabs were cast to bear halfway on the walls, where walls were thick enough to cast and vibrate through a space of half thickness. Slabs resting on 6-in. walls were notched at the edges to permit easy access to the walls below. Exterior walls for the basements were 16 and 12 in. thick and were also cast after the floors were in place with dowels bent from the slabs into the walls.

LIFT SLAB DESIGN

The 12 upper slabs of the two buildings were cast at ground floor level one on top of the other and subsequently lifted into position by hydraulic jacks placed on the tops of the 36 columns. Not only were entire slabs, 70 x 215 ft each weighing 820 tons, raised but they were jacked up in pairs. Thus each jack carried a load of 46 tons. The total weight of the upper slabs which were lifted was 9840 tons.

In the lift slab method of construction it is easiest to use unbraced columns which may be one, two, or three stories high, depending on their diameter and radius of gyration. When the lifting operation begins the usual condition is an unbraced column with the factor of safety close to zero. As the top slab rises, however, the column is braced and when it reaches the top of the column for parking or for permanent attachment, the vertical member has become a pinned column. Al-

though a slab supported by a one-, two-, or three-story column may look precarious, it is actually stable. With the slab finally attached to the column, this type of structure has proved itself resistant to severe earthquake shocks.

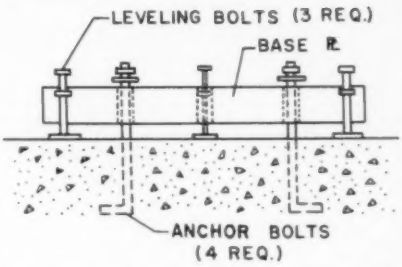

Fig. 2—Anchor bolts and base plate elevation

An unbraced column requires a careful design of the connection between its base and the pier or footing on which it rests. Standard practice is to use a base plate welded to the column and attached to the footing by four anchor bolts (see Fig. 2). These give the column sufficient stability for the symmetrical loading imposed on it by the jacks used during lifting.

In the case of Huron Towers, the base plates were so massive that it was more economical to ship them loose. The four tiers of columns varied in size from 14WF237 to 14WF87. The steel base plates were 4¾ in. x 3 ft 6 in. These were leveled for grouting with three threaded bolts and their position was fixed by four 1¼-in. anchor bolts, 2 ft long. Two holes were provided for grouting while the plate was supported by the three adjustable leveling bolts as shown in Fig. 2.

The ends of the columns were milled to assure uniform bearing on the plates. Attachment was made by welding 8-in. angles to the lower part of the column. The column was attached to the bearing plate with anchor bolts which passed through the angles (see Fig. 3). The un-

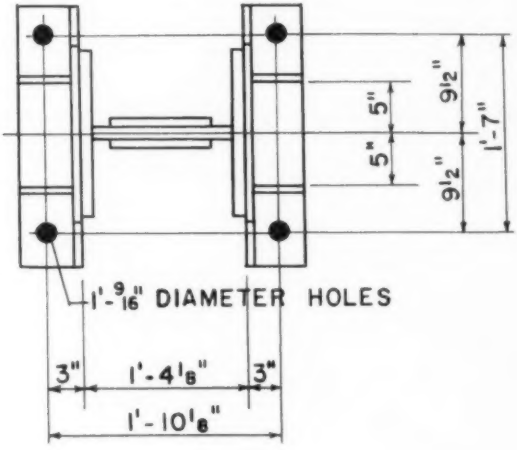

Fig. 3—Column-base plate connection

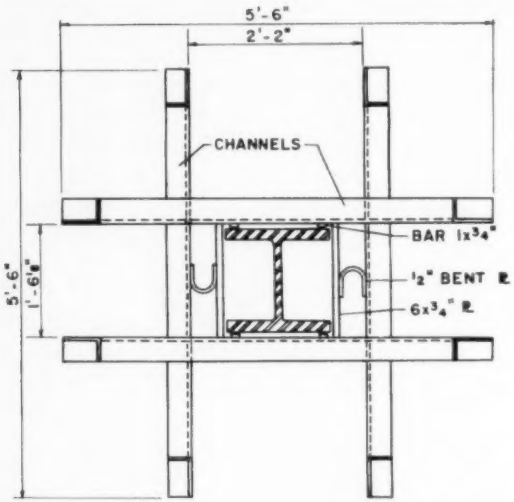

Fig. 4 — Typical column collar

usual size of the columns together with the width of the plate and the four anchor bolts provided sufficient stability for the lifting operation.

Thus, the lower tier of wide flange columns have base angles, shear blocks on the flanges for temporary parking of slab, and plates at the upper end of the web for splicing. The next two tiers have shear blocks at their upper end and splicing plates on the webs at the upper ends. The top tier has only shear blocks at the top end.

The collars cast in the slab made it easy to use either a temporary or a permanent connection with the columns (see Fig. 4 for details of column collar). When the slabs were raised to a position to be attached, two wedges were placed horizontally on top of the shear blocks on each column. The complete connection is shown in Fig. 5. The top of each shear block was slanted exactly the amount of the slant of the wedge. Hence, the top of the wedge was always level. These wedges transmitted the weight from the collars to the shear blocks. They could be tacked into place by a temporary weld or fastened permanently in place by more extensive welding.

When the wedges were welded rigidly in place and connected both to the heavy channels of the collars and to the shear blocks enough continuity was established in the structure to resist wind pressure. Any interior partitions or elevator shafts served to increase the rigidity of the structure.

In designing Huron Towers, prestressing was considered because it permits long slab spans between columns. However, analysis showed that there was nothing to be gained in long spans in the case of an

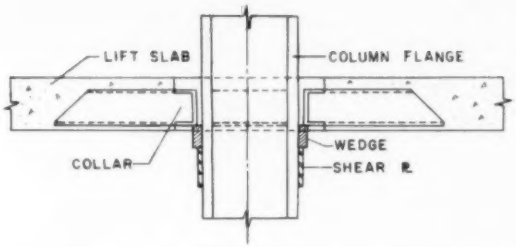

Fig. 5—Detail of column-slab connection

apartment building. Prestressing, in this case would have added to the cost without any compensating advantage.

LIFTING OPERATION

The first step preparatory to a lifting operation is to set up the first tier of columns. In this case, these were three stories high.

The second step is to spray the slab immediately below the last floor to be lifted (in this case the ground or first floor) with a wax, paraffin, or plastic film which prevents the slab from bonding to the slab beneath it.

The third step is to place collars over the columns. These take the shear in the slabs next to the columns, serve as means for attaching the suspension rods that are linked with the jacks, make possible temporary connections so that the slabs may be parked on the columns, and permit the slabs to be welded permanently into place so as to establish continuity between the slabs and columns. For Huron Towers, the collars were welded together from 6-in. car channels. The size of the sections was determined by assuming the concrete bearing on 50 percent of each flange and by stressing the sections to 20,000 psi in both the total load and lifting load conditions. Ends of the channel sections were cut at 45 deg as shown in Fig. 5 to save material. Connections to the columns were made by inserting steel wedges between the bottom of the collars and the top of 12 x 7 in. shear plates welded to the column flanges (see Fig. 6). This same connection was used where slabs were placed in temporary or "parked" positions during the column splicing operation but the wedges were only

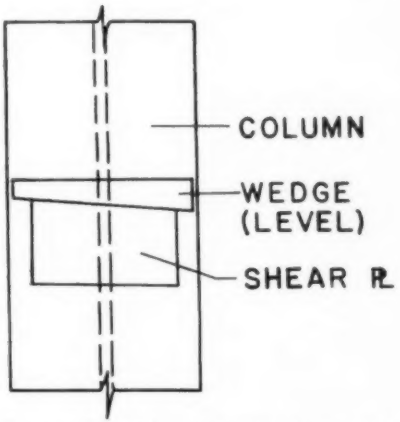

Fig. 6—Typical wedge and shear plate detail

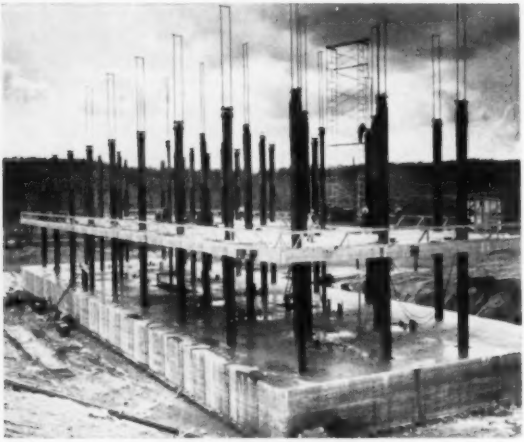

Fig. 7—Lifting operation of first two floors being raised together

securely welded in the final position. It is interesting to note that the bars which terminated opposite column sections were not welded to the collars but were allowed to extend with a hook into the void spaces between the column flanges and the collar. These spaces were finally grouted and adequate anchorage was achieved for the total load, though it was not required for the lifted load.

The fourth step is to place edge forms around the slab and around any openings such as stairwells, elevator, and duct shafts. The reinforcing bars are wired into position the usual way. At this stage electric conduit is fastened in place.

The fifth step is to cast the floor slab, compact it, carefully level it with screeds, and trowel it. These operations are rapid today because power tools do the vibrating, the screeding, and the troweling.

The sixth step is to attach the suspension rods that reach down from the jacks on the top of the columns, to the collars.

The lifting of the slabs was done using 36 jacks which were set up one on the top of each column. The lifting operation was carried out by jacking pairs of slabs as shown in Fig. 7. These were synchronized automatically, and were operated from a central panel or console. The jacks all lifted together because they were connected in series to an electric circuit. This in turn actuated solenoid valves that opened or shut the valves which started or stopped the jacks. If any one jack failed to function, the solenoid could not retract the jack and the red light showed on the panel.

Lift slab construction is safe because a pair of holding nuts actuated by hydraulic motors, are screwed down tight as the jacks raise the slabs. If a jack should burst, which is highly improbable, the slab being raised could not drop but would remain suspended, and the whole

operation would stop automatically until the jack could be replaced. Fig. 8 shows lifting operations on second tier of three-story columns.

The jacks are centered on the columns. They lift a cross arm. Through this pass the suspension rods which have acme thread. The cross arms lift against a pair of nuts. When lifting, these top nuts are stationary. When retracting, these nuts are screwed down automatically by hydraulic motors. On the platform on which the jacks stand are the two lower or holding nuts which have already been described and which constitute a safety lock so that the slabs cannot settle when the jacks retract.

The equipment used on the Huron Towers project is referred to as the automatic short stroke jack. A detail of the jack is shown in Fig. 9. It varies from the older equipment in that the increments of lift are only ½ in. instead of the 3 in. previously used. With the 3-in. lift, synchronization between jacks is controlled by regulating the rate of oil flowing to each jack from a manually operated control panel. Thus, the number of jacks used in a lift is limited by the capacity of the operator to observe and control the regulating valve.

With the short stroke jack there is no attempt made to regulate flow to individual jacks, but rather an oversupply of hydraulic fluid at pressure is supplied to a loop circuit from which is drawn the supply to each jack. The stroke of the jack is limited to ½ in. so that even if one jack lifts completely ahead of all other jacks, it will only advance ½ in. out of level. Practically it is found that this never really takes place as it would require a column with no load for it to happen.

All jacks must complete their ½-in. stroke before the series electrical circuit clears and signals the hydraulic valve to reverse and retract the jacks. All jacks must complete retraction before the circuit again signals the hydraulic valve to reverse and lift again. Progress is in the nature of the inchworm method, although in the case of the Huron Towers project loads of approximately 1000 tons

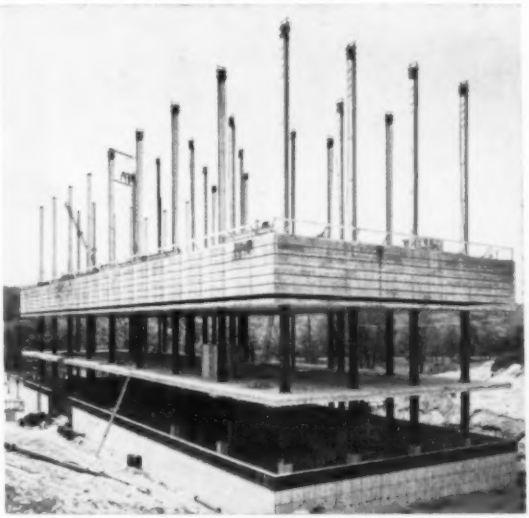

Fig. 8—Lifting operation on second tier of three-story columns

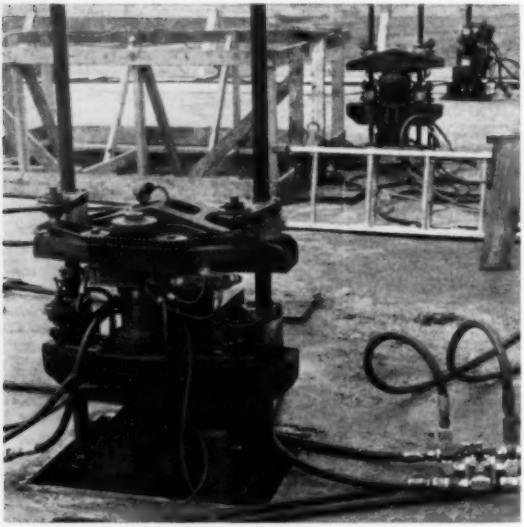

Fig. 9—Detail of automatic short stroke jack in position on top of the column during lifting operation

lifted on 36 jacks progressed as fast as 15 ft per hr on a maximum trial rate. The usual speed was about 9 ft per hr. A view of the jacks in operation is shown in Fig. 9.

Since control is automatic the operator is relieved from monotonous duty at the control panel and is better able to supervise and plan the work schedule. Night lifting is readily performed without flood lighting as signal lights are provided on each jack to indicate which jack might have stopped the lift in the event that a complete stroke was not accomplished. Slab flatness is positive with this type of control especially on larger slabs containing more columns. Fig. 10 shows the completed structure after all slabs have been lifted and connected to the columns.

SUMMARY

The advantages of the lift slab method of construction are numerous. In the case of Huron Towers, the slabs could be placed in a stack during winter months under the protection of a plastic tent (see Fig. 11). No formwork and shoring are needed except for the edges of the slab. The undersides of the slab are as smooth as the floors and need no plastering. No time is lost waiting for the slabs to cure because usually as soon as the jacks are in position ready to lift, the concrete is hard enough to raise. The lifting operation can be speeded up as on the Huron Towers job by raising the slabs in pairs (shown in Fig. 7 and 8). Perhaps the greatest recommendation of the new method is that it saves completion time which means money. Huron Towers would have risen more rapidly if it had not been for the steel strike and then a strike of structural steel erectors.

Traditionally, architecture has combined three structural components, viz., walls, columns, and floors. This trinity is to be found in buildings all over the world. Each culture and each period in history creates a

characteristic style of architecture, but we invariably find the same three essential units in all types of building.

Critics were shocked, therefore, when architects began a few decades ago to erect modern structures omitting walls altogether or at least relegating them to a minor function. Steel frame buildings which began to be common about the turn of the century eliminated the need for heavy masonry bearing walls. Steel columns took their place and the wall became only a means of separating the indoors from the outdoors, an essential function but one that could be filled by thin panels which were properly weatherproofed and insulated. Since such dividers of space could be supported by a steel beam at each floor, they could be made of glass or sheet metal or light masonry units. Similarly reinforced concrete frame construction dispensed with the need for structural walls. But like the ivy which clings to the walls of ruined buildings after they have lost their strength, architects continued to regard vertical expanses of masonry as essential long after the functional use of walls had disappeared.

The most recent step toward reducing structures to columns and floors is lift slab construction. Characteristically this employs steel columns and flat reinforced concrete floors though it can also be used with reinforced concrete columns and coffered ceilings where a thick floor structure is needed. During erection a lift slab building demonstrates most dramatically the emancipation of architecture from walls. One sees thin floors apparently floating one above the other. When such structures are enclosed, they often lose their appearance of light-

Fig. 10—Lift slab operation completed for one of the towers

Fig. 11 — Plastic tent for winter protection of lift slabs

ness, unless the architect is a skillful designer and takes pains to express the column and slab relationship.

The "modern plastic" which made these new wall-less towers possible is concrete. Its compressive strength, its versatility, its economy, its fire protection, and its durability metamorphosed a program into an actuality.

ORGANIZATION

The developers of Huron Towers apartments were Morton R. Scholnick and his associate, Seymour Dunitz, both of Detroit. The estimated cost of the project is $7,000,000 and of this the U.S. Government furnished $5,770,000 through the Federal Housing Administration. King and Lewis, Detroit, were the architects. The consulting engineers were R. H. McClurg and Associates, Detroit. The Long Construction Co., Kansas City, Mo., was the general contractor. This firm operates under a license from the United States Lift Slab Corp., Kansas City.

ACKNOWLEDGMENTS

The author wishes to express his appreciation to Marshall Long, of Long Construction Co., Harry King, of King and Lewis, and Richard McClurg of R. H. McClurg and Associates for their assistance in preparing this paper.

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Title No. 57-68

Camber in Prestressed Concrete Beams

By D. E. BRANSON and A. M. OZELL

Investigation examined experimentally the initial-plus-time-dependent camber deformation in both noncomposite and composite prestressed concrete beams. Methods for calculating these deformations relative to certain properties of the concrete are presented. Ten noncomposite beams and five composite beams were used in the tests. Total camber deformation was found to follow closely the prestress level (relative to concrete strength) but to be relatively insensitive to different concrete strengths and atmospheric conditions. Total camber appears to reach its ultimate value relatively early (100 to 200 days for the test beams) compared to the long-time shrinkage and creep strains that occur. The analytical methods set forth for predicting camber deformation in both noncomposite and composite beams were found to be in good agreement with the test results. The necessary concrete coefficients for the analysis of total camber were experimentally determined and are presented here and recommended for design purposes.

■ IN PRACTICE IT HAS BEEN FOUND that initial camber generally exceeds the expected or computed camber when the latter is based on the cylinder modulus of elasticity. One of the main reasons for this is that the elastic and creep strains occur together during the stressing operation and it is virtually impossible, from a practical standpoint, to obtain a purely elastic or initial camber. Also, the rate of concrete creep is rapid immediately following prestressing, so that the camber does not just occur initially and stop, but is rather a progressive behavior from the start.

Generally speaking, ultimate values of the ratio of creep strain to elastic strain (C_u) of 1.0, 2.0, and 3.0 correspond to relative humidities of approximately 100, 70, and 50 percent, respectively. The ACI-ASCE Committee Recommendations[1] provide for general time-dependent camber calculations by allowing the concrete strains to increase ultimately from 100 to 300 percent ($C_u = 1.0$ to 3.0), depending on atmospheric conditions. This is seen to be the general approach to the problem without specific provision for the effect of different stress levels, prestress loss, increase in concrete stiffness with age, differential shrinkage, and additional slab resistance to camber growth for composite beams.

Even if the creep coefficient C (ratio of creep strain to elastic strain) under constant uniform sustained stress is known, the stress distribution is nonuniform and the sustained stress magnitude varies because of the loss of prestress. This paper includes a study of these and other factors that influence the initial-plus-time-dependent camber deformation in both noncomposite and composite beams.

Notation

1 = subscript denoting the cast-in-place slab of a composite beam

2 = subscript denoting the precast stem of a composite beam

3 = area

B = camber coefficient defined as the ratio of inelastic camber to elastic camber

C = creep coefficient defined as the ratio of creep strain to elastic strain

C_v = creep coefficient for the variable sustained stress in a prestressed concrete beam

c = distance from the centroid of the area of a beam cross section to the extreme fiber

c = subscript denoting concrete

d = depth of a beam cross section

E = modulus of elasticity

E_{sus} = reduced or sustained modulus defined as the ratio of concrete stress to elastic-plus-creep strain

e = eccentricity (e_e at beam end and e_m at midspan)

ε = unit strain

F = differential shrinkage force

f_c' = concrete strength at 28 days

f'_{ci} = concrete strength at the time of prestressing (for the experimental work included herein, all prestressing was done at 28 days)

H = slope of strain distribution diagram

I = moment of inertia (second moment of the area)

i = subscript denoting an initial value

k = prestress loss coefficient defined as the ratio $(P_i - P_t)/P_i$

L = beam span (c-c of supports)

M = bending moment

M' = bending moment resulting from slab dead load

P = prestress force

p = subscript denoting the time of prestressing

q = subscript denoting the time of slab casting

R = radius of curvature

θ = rotation

S = summation coefficient defined by Eq. (6)

σ = unit stress

t = subscript denoting variable time measured from time of prestressing

t' = subscript denoting variable time measured from time of slab casting

u = subscript denoting the time beyond which there is negligible change; i.e., the ultimate value

w = uniformly distributed load

w' = uniformly distributed slab dead load

X = measured camber

x = variable distance along a beam

Y = computed camber using the summation coefficient S_t

y = variable deflection of a beam

Z = computed camber using the slope of the measured strain distribution diagram

$B'_{t'}$, $C'_{vt'}$, $S'_{t'}$ = notation referring to the increase in the respective coefficients for time t' (following slab casting), but where the sustained stress was applied at the time of prestressing

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ACI member **A. M. Ozell**, Professor of Civil Engineering, University of Florida, Gainesville, has been engaged in prestressed concrete research for the past 6 years. Dr. Ozell has written numerous papers, many of them appearing in the ACI JOURNAL, reporting the results of his static and fatigue tests, and other research activities.

DESCRIPTION OF SPECIMENS AND TEST DATA

This study was conducted on five pairs of beams. Each pair consisted of one composite beam and one bare beam. Shrinkage specimens and control cylinders were used for each type of concrete. Table 1 is a summary of information for the beams.

All ten beams were post-tensioned at 28 days with three straight unbonded ¾ in. steel bars. The beam dimensions were 8 in. x 12 in. x 20 ft, and the slabs were 3 in. x 26 in. x 20 ft, as shown in Fig. 1. The steel bars had a modulus of elasticity of 26.38×10^6 psi and a yield strength (0.2 percent offset) of 147,000 psi.

A mechanical strain gage (10 in. gage length) was used to measure the concrete strains. The gage points were steel inserts imbedded in the concrete. Each beam had four gages distributed from top to bottom (see Fig. 1) at both ends and midspan on both sides. Thus a total of 24 gages on each beam provided the data for determining the shrinkage and creep strains in the beams. Dial gages were used on both sides of each beam at midspan to measure the total camber deformation. An average of the two readings is reported for each beam. The accuracy of the dial gages (0.001 in.) for measuring total camber values of up to 1 in. and higher provided excellent data for this part of the study. Camber measurements for the composite beams at midspan were obtained using a Dumpy level and scale providing readings to 0.002 in. Additional data obtained included temperature and relative humidity and the strength and elastic properties of the concrete as determined by 6 x 12 in. cylinder tests.

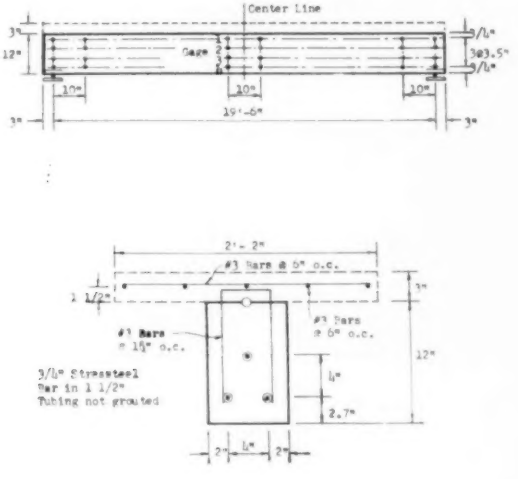


Fig. 1 — Bare beam with cast-in-place slab shown dotted

TABLE 1 — BEAM DATA

Beam No.	Concrete strength 28 days, psi	Stress, percent f'_{ci}	Level, psi	Location	Composite beam No.	Time between prestressing and slab casting, days
1,2	5030	25	1250	Lab	1-C	101
3,4	5030	45	2250	Lab	4-C	101
5,6	5030	65	3250	Lab	6-C	101
7,8	3760	60	2250	Lab	8-C	37
9,10	3760	60	2250	Field	10-C	93

A summary of the test beam data is given in Table 2. Fig. 2 and 3 show some of the specimens and laboratory testing site. The shrinkage data for the beams are shown in Fig. 4. Typical curves showing the elastic-plus-creep strains (for Beam 4) at four gage levels (for both the beam end and midspan sections) are presented in Fig. 5, and the corresponding strain distribution diagrams are shown in Fig. 6. The total midspan camber versus time curves for the ten beams are shown in Fig. 7.

CAMBER OF NONCOMPOSITE BEAMS

The initial curvature expression for any prestressed concrete beam (symmetrical or unsymmetrical with respect to the horizontal centroidal axis) under the action of the beam dead load and the prestress force is given by

$$\frac{d^2y}{dx^2} = \frac{1}{E_p I}(M_x - P_i e_x) \quad \text{.....(1)}$$

where E_p is Young's modulus at the time of prestressing, and P_i is the initial prestress force. For ranges of working stress in the type of concrete ordinarily used for prestressed concrete, the idealized stress-strain curve shown in Fig. 8 for an instantaneous and then constant sustained stress is closely realized. It follows that for a constant sus-

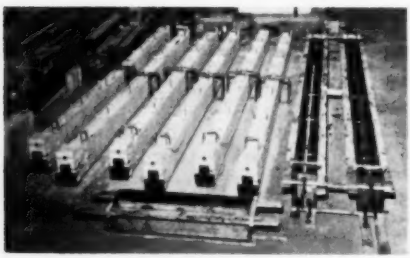


Fig. 2—Laboratory site showing Beams 1-6, shrinkage specimens, dial gage frames, and forms for Beams 7 and 8

Fig. 3 — Four composite beams in laboratory with noncomposite beams alongside

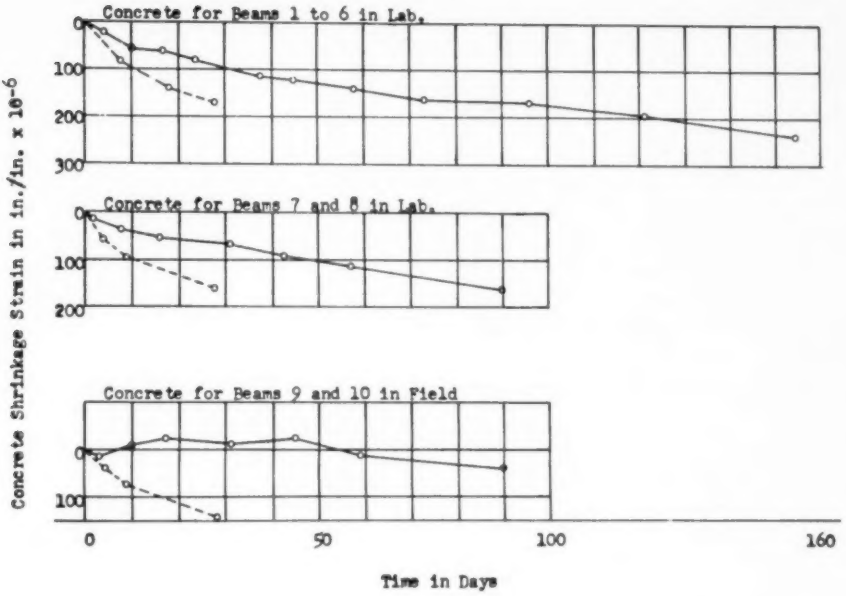


Fig. 4—Concrete shrinkage versus time for precast beam concretes. Each plotted point is an average of 16 readings which indicated a uniform shrinkage strain distribution for the size members used. The dotted curves show the concrete shrinkage from the casting day up to 28 days. Time zero is 28 days after casting at which time the prestress was applied

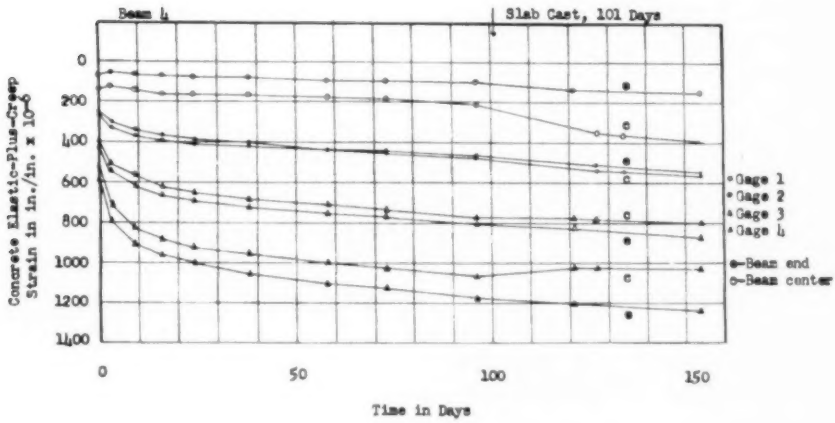


Fig. 5—Concrete elastic-plus-creep strains (Beam 4). Ultimate shrinkage distribution: 9 days—5, 31 days—99, 90 days—167, 154 days—237 in. per in. x 10^{-6}

TABLE 2 — SUMMARY OF DATA FOR THE TEN PRESTRESSED CONCRETE BEAMS

Beam No.	Bar No. (1 • 2 3 • •)	Jack load, kips	Actual load using electrical gages,* kips	Actual total load P, kips	Measured eccentricity at beam end e_e, in.	Actual eccentricity at midspan e_m, in.	Total Pe_e, in.-kips	Total Pe_m, in.-kips	Actual eccentricity of center of gravity of section $e = Pe/P$†		Initial computed stresses				σ_{max}‡ percent f'_{ci}
											End		Midspan		
									End, in.	Midspan, in.	σ_T, psi	σ_B, psi	σ_T, psi	σ_B, psi	
1	1	21	18.6	57.9	−0.56	−0.41	125.0	133.5	2.16	2.31	+ 28	−1352	−250	−1059	26.9
	2	21	19.8		3.41	3.56									
	3	21	19.5		3.48	3.62									
2	1	21	22.1	65.1	−0.62	−0.47	131.6	140.9	2.02	2.16	− 17	−1470	−291	−1182	29.2
	2	21	20.8		3.38	3.52									
	3	21	22.2		3.38	3.52									
3	1	36	33.2	101.5	−0.49	−0.25	223.4	247.8	2.20	2.44	+ 73	−2394	−119	−2190	47.5
	2	36	33.0		3.51	3.75									
	3	36	35.3		3.51	3.75									
4	1	36	37.2	99.9	−0.60	−0.25	190.6	225.9	1.91	2.26	− 86	−2190	−219	−2048	43.6
	2	36	31.0		3.38	3.75									
	3	36	31.7		3.41	3.75									
5	1	50	49.7	142.1	−0.60	−0.25	283.8	334.1	2.00	2.35	− 55	−3188	−106	−3130	63.5
	2	50	47.7		3.38	3.75									
	3	50	44.7		3.41	3.75									
6	1	50	46.7	139.5	−0.65	−0.25	283.3	336.4	2.03	2.41	− 28	−3156	− 64	−3112	63.8
	2	50	46.6		3.41	3.75									
	3	50	46.2		3.35	3.75									
7	1	36	35.7	93.6	−0.51	−0.25	184.5	208.2	1.97	2.22	− 48	−2083	−242	−1875	55.4
	2	36	29.2		3.52	3.75									
	3	36	28.7		3.48	3.75									
8	1	36	26.2	87.4	−0.51	−0.25	200.6	222.9	2.30	2.55	+108	−2106	− 95	−1891	56.0
	2	36	28.7		3.48	3.75									
	3	36	32.5		3.51	3.75									

9	1	36	30.1	90.0	−0.34	−0.25	209.8	217.5	2.33	2.41	+129	−2187	−154	−1890	58.3
	2	36	30.0		3.70	3.75									
	3	36	29.9		3.63	3.75									
10	1	36	28.5	91.6	−0.28	−0.25	220.6	229.6	2.41	2.51	+170	−2265	−106	−1975	60.4
	2	36	30.2		3.54	3.75									
	3	36	32.9		3.70	3.75									

*These values were used in the calculations.
†The midspan eccentricity is greater than at the beam end because of camber.
‡The 28-day cylinder strengths were 5030 psi for Beams 1-6 and 3760 psi for Beams 7-10. The beams were prestressed at 28 days.

tained prestress force, the curvature expression after any time t would be given by

$$\frac{d^2y}{dx^2} = \frac{1 + C_t}{E_p I}(M_x - P_i e_x) \quad \text{(2)}$$

where C_t is the creep coefficient corresponding to the time t.

Now consider the effect of the variable sustained stress resulting from the loss of prestress and introduce the prestress loss coefficient $k_t = (P_i - P_t)/P_i$.

This is an important consideration since the initial prestress force is usually reduced from 15 to 30 percent. Eq. (2) is now written as

$$\frac{d^2y}{dx^2} = \frac{1 + C_{v_t}}{E_p I}(M_x) - \frac{1}{E_p I}(P_i e_x) + \left[\sum_{n=0}^{t} \frac{k_n - k_n - 1}{E_n}\right](P_i e_x)\frac{1}{I} - \left[\sum_{n=0}^{t}(1 - k_n)(C_{v_n} - C_{v_{n-1}})\right](P_i e_x)\frac{1}{E_p I} \quad \text{(3)}$$

where C_{v_t} is the creep coefficient for the variable sustained stress for any time t and n is any feasible increment of time. The first term is the elastic-plus-inelastic curvature expression for the dead load moment M_x. This bending moment does not vary with time. The second term is the elastic curvature expression for the initial prestress force. The third term is the curvature expression for the elastic deflection resulting from the loss of prestress $k_n P_i$. It is necessary to express this in a form requiring the summation of the incremental values of k_n/E_n since the history of the stress loss factor k_t and the concrete stiffness E_t must be taken into account. The last term is the curvature expression for the inelastic camber deformation involving the actual prestress force $(1 - k_n)P_i$ and the effective creep coefficient ΔC_{v_t} during the increment of time. The history of the actual prestress force and creep coefficient under variable sustained stress is thus taken into account.

Notice that, in the first and last terms which contain the actual creep coefficient

under the variable stress, Young's modulus at the time of prestressing is used. The reason for this is that the actual experimentally determined creep coefficient would include any inelastic effect of the concrete increasing in stiffness with time.

Eq. (3) can be rewritten in the form

$$\frac{d^2y}{dx^2} = \frac{1}{E_p I}\left[(1 + C_{v_t}) (M_x) \right] + \frac{1}{E_p I}\left[-1 + \sum_{n=0}^{t} \frac{k_n - k_{n-1}}{E_n/E_p} - \sum_{n=0}^{t} (1 - k_n) (C_{v_n} - C_{v_{n-1}}) \right](P_i e_x) \quad \text{(4)}$$

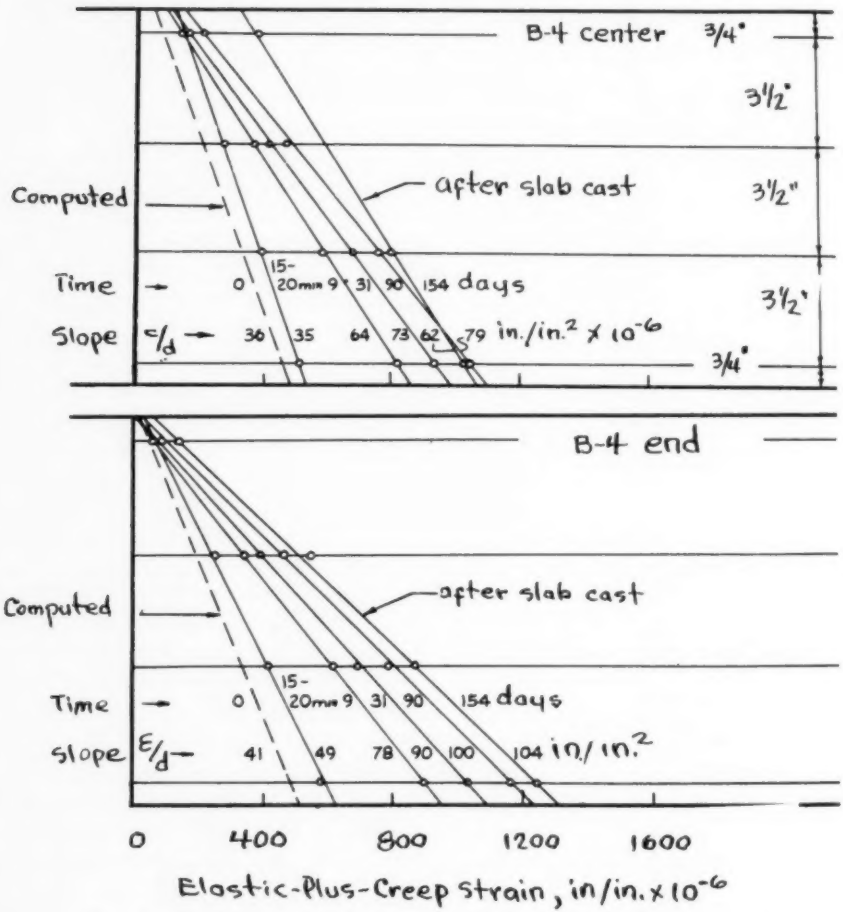


Fig. 6—Elastic-plus-creep-strain distribution diagrams (Beam 4)

which reduces to Eq. (2) when the prestress loss is zero. For convenience Eq. (4) is written as

$$\frac{d^2y}{dx^2} = \frac{1}{E_p I}\left[(1 + C_{v_t}) (M_x) - (1 + S_t) (P_i e_x) \right] \quad \ldots\ldots (5)$$

where

$$S_t = -\sum_{n=0}^{t} \frac{k_n - k_{n-1}}{E_n/E_p} + \sum_{n=0}^{t} (1 - k_n) (C_{v_n} - C_{v_{n-1}}) \quad \ldots\ldots (6)$$

The coefficient S_t will be referred to as the summation coefficient.

The solution of Eq. (5) for midspan camber of simply supported prestressed concrete beams is given by the following two equations: For a straight cable

$$y = \frac{1}{E_p I}\left[(1 + S_t) P_i L^2 e/8 - (1 + C_{v_t}) \, 5wL^4/384 \right] \quad \ldots\ldots (7)$$

For a parabolic cable

$$y = \frac{1}{E_p I}\left[(1 + S_t) \frac{P_i L^2}{48} (e_o + 5e_m) - (1 + C_{v_t}) \, 5wL^4/384 \right] \quad \ldots\ldots (8)$$

It is seen that the summation coefficient S_t corresponds to a creep coefficient but would always be somewhat smaller. This can best be shown by referring to Eq. (6). The factor $(C_{v_n} - C_{v_{n-1}})$ is equal to C_{v_t} for any time $n = t$, and the factor $(1 - k_t)$ is always less than 1.0, except at $t = 0$. Therefore S_t is always less than C_{v_t}, except when

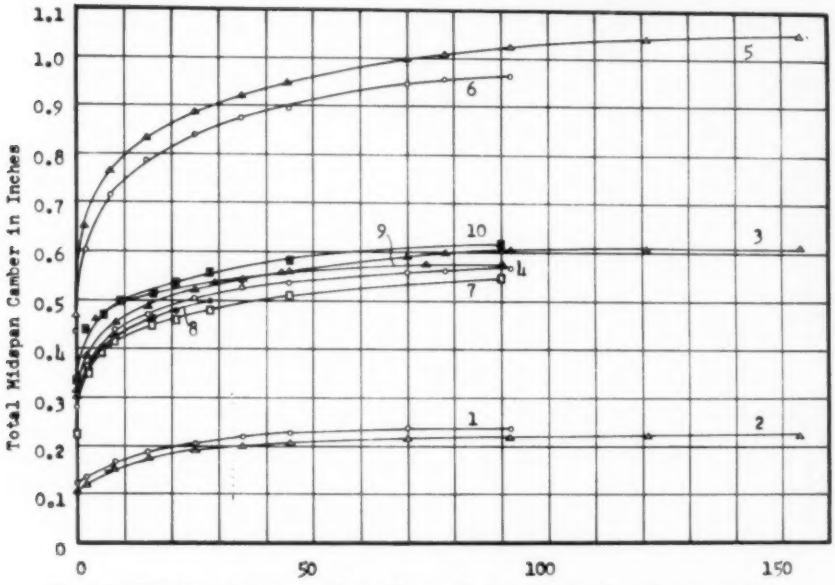


Fig. 7—Total midspan camber (from dial gage readings) versus time

both are zero. Also the first summation term in Eq. (6) is the elastic deflection resulting from the loss of prestress which further reduces S_t.

A second (more direct but less general) approach to the solution of camber deformation in prestressed concrete beams is found by referring to Eq. (2), in which the prestress force was assumed to be constant. The solution of this equation is

$$y = (1 + C_t) \iint \frac{M_x - P_i e_x}{E_p I} \, dx \, dx$$

$$= (1 + C_t) \, y_i$$

where y_i is the computed initial elastic camber. It is apparent then that the creep coefficient is actually a camber coefficient B_t, which is defined at the ratio of inelastic camber to elastic camber. Hence

$$y = (1 + B_t) \, y_i \qquad (9)$$

or in basic form

$$\frac{d^2y}{dx^2} = \frac{1 + B_t}{E_p I} (M_x - P_i e_x) \qquad (10)$$

Since the theoretical solution given by Eq. (5) requires experimentally determined time-dependent data, this coefficient B_t could be found and used more readily with Eq. (10). The camber coefficient

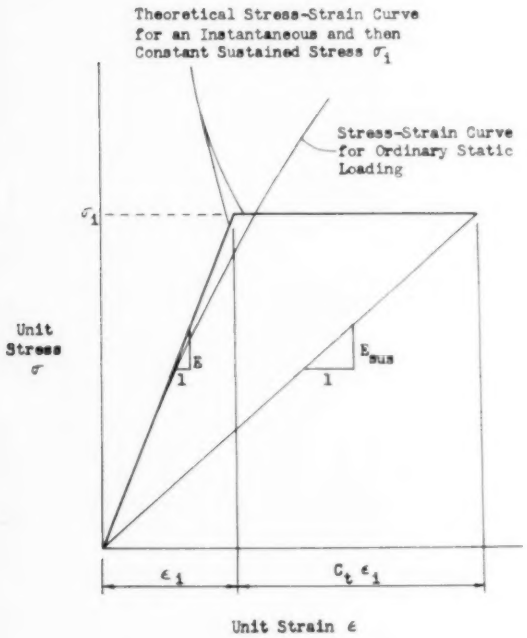


Fig. 8 — Idealized concrete stress-strain curve

B_t is seen to be primarily a property of the concrete (resulting from creep) but includes the effect of prestress loss as well. It would therefore be expected to be smaller than the corresponding creep coefficient as the curves for the two coefficients C_{v_t} (Fig. 9) and B_t (Fig. 12) verify.

The basic camber equation, Eq. (5), which includes Eq. (6) for the coefficient S_t, provides an explanation for the principal factors that affect camber, but these equations are not practical for design purposes covering wide ranges of conditions. However, with the data included herein for the coefficient S_t, the camber deformation can be deter-

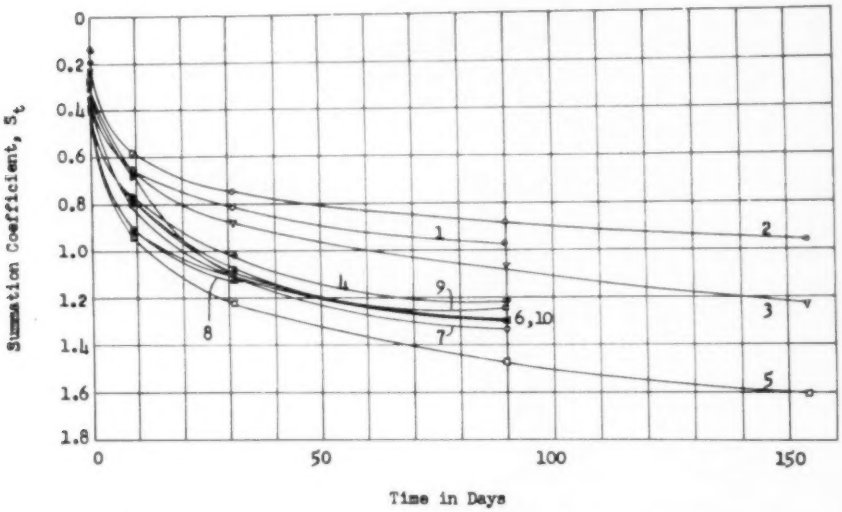


Fig. 9—Creep coefficient under variable sustained stress, C_{v_t} versus time

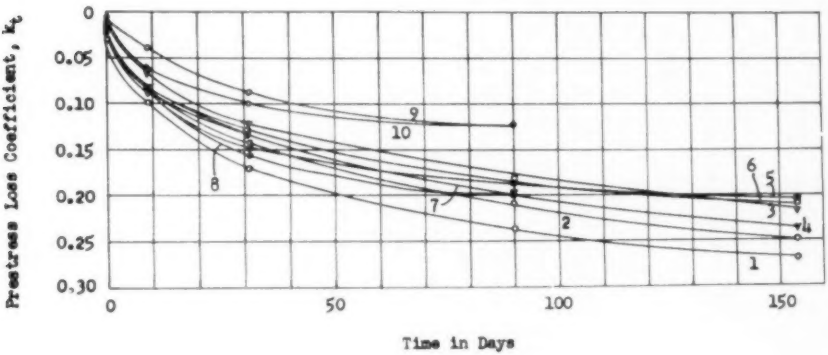


Fig. 10—Prestress loss coefficient, k_t versus time

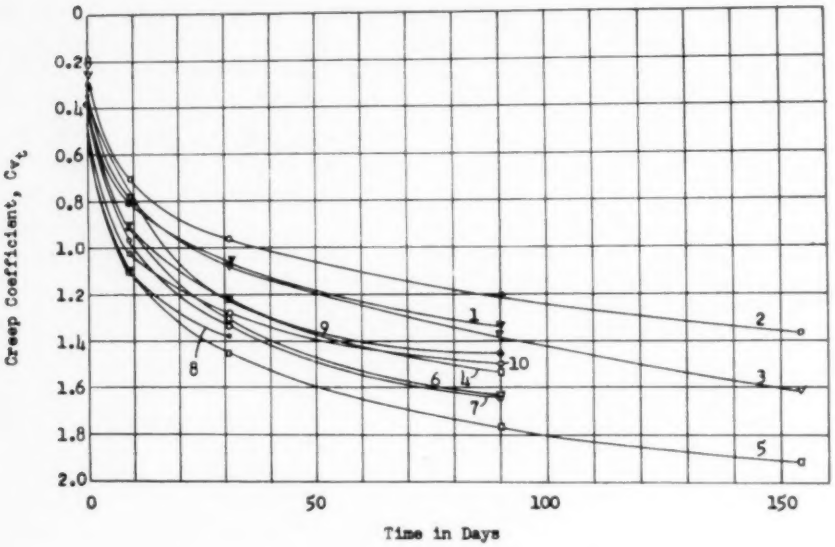


Fig. 11—Summation coefficient, S_t versus time

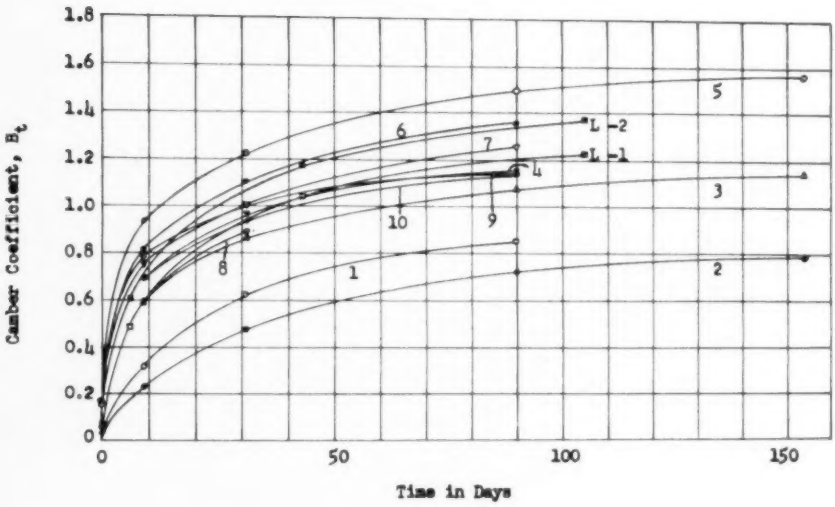


Fig. 12—Camber coefficient, B_t versus time

mined easily with Eq. (7) or (8) for the conditions of these tests. The values for S_t were found to be somewhat less than the creep coefficients C_{v_t} of the corresponding beams, since S_t includes the effects of loss of prestress (see Fig. 9 and 11).

Eq. (10) provides a direct solution to the camber problem using the camber coefficient B_t, which also includes the effect of prestress loss. The values for B_t (Fig. 12) were found to be smaller than the corresponding creep coefficients C_{v_t}.

CAMBER OF COMPOSITE BEAMS

Consider first the case of any sustained load applied to a prestressed concrete beam sometime after prestressing. The strain distribution would be altered, thus requiring a modification of the equations developed in the previous section. Eq. (5) would become

$$\frac{d^2y}{dx^2} = \frac{1}{E_p I}[\,(1 + C_{v_t})\,(M_x) - (1 + S_t)\,(P_i\,e_x)\,] + \frac{1 + C_{v_{t'}}}{E_q I}\,(M_x') \quad (11)$$

where $C_{v_{t'}}$ is the appropriate creep coefficient for the concrete subjected to sustained stress at the age the additional load was applied, E_q is Young's modulus for the concrete at the age the load was applied, and M_x' is the moment caused by the additional sustained load. Previous observations showed that the creep of concrete is less for sustained loads applied at increasing ages, and hence the creep coefficient $C_{v_{t'}}$ varies depending on the age at which the additional sustained load was applied.

Now consider the special but more practical case of the cast-in-place slab placed to form a composite beam at some date after prestressing. Eq. (11) would apply if the slab were not bonded to the prestressed beam. But since it is, the slab will provide resistance to camber deformation in addition to inducing a downward deflection as a result of the dead load and differential shrinkage.

In considering the strains resulting from creep and differential shrinkage, it is seen that these strains are caused by internal effects and not by any external moment. Hence the composite or transformed section properties are not effective in resisting these strains as a unit. For example, if the stress were zero at the top of the stem of the composite beam, there would be no tendency for the slab to shorten directly as a result of creep in the prestressed stem. The only resistance to camber that it would have would be a bending resistance depending only on the slab *EI* value. The stem *EI* value is a part of the initial strain expression already included. This same concept of separate and not composite resistance also enters into the differential shrinkage theory.

All but the last term of Eq. (11), on the right hand side, is the slope of the elastic-plus-creep strain distribution diagram at the time t, caused by the loading that existed at the time of prestressing. The last term is the decrease in this slope as a result of the additional loading.

Since the problem in the composite beam is to determine the total camber occurring before and after slab casting, Eq. (11) must be modified to include the slab resistance to further bending as a result of creep in the prestressed stem as well as differential shrinkage. This resistance is brought about through a decrease in the creep strains that would occur if there were no bond between the stem and slab. If the differential shrinkage effect is temporarily neglected, the expression for curvature corresponding to the total camber for a composite beam would be

$$\frac{d^2y}{dx^2} = \left[\frac{1}{E_p I_2}(1 + C_{v_q})(M_x) - \frac{1}{E_p I_2}(1 + S_q)(P_i e_x) + \frac{1}{E_q I_2}(M_x')\right]$$

$$+\left[\frac{C'_{v_{t'}}}{E_p I_2}(M_x) - \frac{S'_{t'}}{E_p I_2}(P_i e_x) + \frac{C_{v_{t'}}}{E_q I_2}(M_x')\right]\left[\frac{E_2 I_2}{E_1 I_1 + E_2 I_2}\right] \quad (12)$$

where $C'_{v_{t'}}$ and $S'_{t'}$ are the increase in the coefficients for the time t', but where the sustained stress was applied at the time of prestressing, and q is the time of slab casting. That is,

$$C_{v_t} = C_{v_q} + C'_{v_{t'}} \quad \text{and} \quad S_t = S_q + S'_{t'}$$

The last bracket of Eq. (12) includes the effect of slab resistance since the EI values are the resistance factors for bending. The sum of E_1I_1 and E_2I_2 is used here rather than the EI of the transformed composite section because the continued camber (or deflection) resulting from creep in the precast stem is not resisted by the composite section as a unit. But the two Young's moduli, E_1 and E_2, for the slab and stem, respectively, are increasing with time. However, these two moduli appear in ratio form in the last bracket of Eq. (12) so that the use of the 28-day cylinder moduli for both E_1 and E_2 is sufficient here.

When I_1 becomes zero (no slab), Eq. (12) reduces to Eq. (11). When I_1 increases without bound, but M_x' is finite (deep cast-in-place element), Eq. (12) would show no increase in camber beyond that which occurred up to and immediately following slab casting since the slab dead load factor is in the first bracket to account for this deflection.

The additional decrease in curvature (camber) after the slab has been cast, as a result of the differential shrinkage force F, is given by the expression*

*This was presented more fully in the discussion by the authors of "Differential Shrinkage in Composite Beams," by Halvard W. Birkeland, ACI JOURNAL, V. 32, No. 6, Part 2, Dec. 1960 (*Proceedings* V. 56), pp. 1529-1546.

$$\left[\frac{1 + C_{v_{t'}}}{E_q I_2}\,(g F c_{2T})\right]\left[\frac{E_2 I_2}{E_1 I_1 + E_2 I_2}\right]$$

where g is a factor, less than 1.0, to account for the fact that the total force F does not act over the entire time t'. The force F may be assumed here to remain constant over the entire length of the beam although this is not quite correct.

Because of the general parabolic variation of the differential shrinkage force with time, g is assumed to be 2/3. Hence, including the effect of differential shrinkage, Eq. (12) becomes

$$\frac{d^2y}{dx^2} = \left[\frac{1}{E_p I_2}(1 + C_{v_q})\,(M_x) - \frac{1}{E_p I_2}(1 + S_q)\,(P_i e_x) + \frac{1}{E_q I_2}(M_x')\right]$$

$$+\left[\frac{C'_{v_{t'}}}{E_p I_2}(M_x) - \frac{S'_{t'}}{E_p I_2}(P_i e_x) + \frac{C_{v_{t'}}}{E_q I_2}(M_x') + \frac{1 + C_{v_{t'}}}{E_q I_2}(\tfrac{2}{3} F c_{2T})\right]$$

$$\left[\frac{E_2 I_2}{E_1 I_1 + E_2 I_2}\right] \qquad (13)$$

It follows that Eq. (10) for the composite beam would become

$$\frac{d^2y}{dx^2} = \left[\frac{1 + B_q}{E_p I_2}(M_x - P_i e_x) + \frac{1}{E_q I_2}(M_x')\right]$$

$$+\left[\frac{B'_{t'}}{E_p I_2}(M_x - P_i e_x) + \frac{B_{t'}}{E_q I_2}(M_x') + \frac{1 + B_{t'}}{E_q I_2}(\tfrac{2}{3} F c_{2T})\right]$$

$$\left[\frac{E_2 I_2}{E_1 I_1 + E_2 I_2}\right] \qquad (14)$$

Eq. (13) and (14) are also applicable when the precast beam is either symmetrical or unsymmetrical with respect to the horizontal centroidal axis.

The solutions for Eq. (13) and (14) are, respectively,

$$y = \left[\frac{1}{E_p I_2}(1 + S_q)\frac{P_i L^2}{48}\,(e_o + 5e_m) - \frac{1}{E_p I_2}(1 + C_{v_q})\,5wL^4/384\right.$$

$$\left. - \frac{1}{E_q I_2}\,5w'L^4/384\right] + \left[\frac{S'_{t'}}{E_p I_2}\,\frac{P_i L^2}{48}\,(e_o + 5e_m) - \frac{C'_{v_{t'}}}{E_p I_2}\,5wL^4/384\right.$$

$$\left. - \frac{C_{v_{t'}}}{E_q I_2}\,5w'L^4/384 - \frac{1 + C_{v_{t'}}}{E_q I_2}\,\tfrac{2}{3} F c_{2T}\,\frac{L^2}{8}\right]\left[\frac{E_2 I_2}{E_1 I_1 + E_2 I_2}\right] \qquad (15)^*$$

*See Table 4 for an explanation of the terms of Eq. (15).

and

$$y = \left[\frac{1 + B_q}{E_p I_2} \frac{P_i L^2}{48} (e_o + 5e_m) - \frac{1 + B_q}{E_p I_2} 5wL^4/384 - \frac{1}{E_q I_2} 5w'L^4/384 \right]$$

$$+ \left[\frac{B'_{t'}}{E_p I_2} \frac{P_i L^2}{48} (e_o + 5e_m) - \frac{B'_{t'}}{E_p I_2} 5wL^4/384 - \frac{B_{t'}}{E_q I_2} 5w'L^4/384 \right.$$

$$\left. - \frac{1 + B_{t'}}{E_q I_2} \frac{2}{3} F c_{2T} \frac{L^2}{8} \right] \left[\frac{E_2 I_2}{E_1 I_1 + E_2 I_2} \right] \quad \text{(16)}$$

Both of these equations provide the solution for midspan camber of composite beams. Eq. (15) involves the use of the summation coefficient S and the creep coefficient C_v while Eq. (16) includes the camber coefficient B.

EXPERIMENTALLY DETERMINED COEFFICIENTS

The values of the creep coefficient C_{v_t} were determined from the strain distribution diagrams for the total measured-strain-less-shrinkage strain. The strains at the bottom fiber were used. Only the average of the end and midspan values are reported since these were practically the same. The elastic strain resulting from loss of prestress was discounted to obtain the true creep coefficient. The actual values for the creep coefficient were based on the computed initial elastic strain. This was found to be necessary to correlate all of the theoretical and experimental camber data. The values for the creep coefficient C_{v_t} versus time are shown in Fig. 9 for the test beams.

The loss of prestress was determined from the concrete shrinkage-plus-creep strains (at the beam ends) at the level of each bar using the strain distribution diagrams, and also from direct strain readings on the bars themselves using electrical gages. There was reasonably close agreement between the two methods. The precentage loss of prestress (in decimal form as the prestress loss coefficient k_t) versus time curves are shown in Fig. 10, for which an average of the three bars were used for each beam.

The effect of prestress loss on camber can best be described by referring to the summation coefficient S_t, which was defined by Eq. (6). The first term is the elastic deflection resulting from the loss of prestress, and the second term is the creep effect with the variable prestress force given by $(1 - k_t) P_i$ for any time t. Hence the summation coefficient S_t, involving the time-dependent variables of percentage prestress loss k_t, concrete stiffness $E_{t'}$ and the creep coefficient $C_{v_{t'}}$ corresponds to the creep coefficient for the constant sustained moment M_x. It therefore includes the total time-dependent effect on camber for

the prestress moment $P_i e_x$. It is seen from the curves in Fig. 11 that each coefficient S_t has approximately the same relative position as the corresponding creep coefficient C_{v_t} (Fig. 9), but the S_t values are smaller in every case because of the effect of prestress loss.

The camber coefficient B_t was defined as the ratio of inelastic camber to elastic camber. The values for the camber coefficient were determined by dividing the total measured camber minus the initial theoretical elastic camber (using the cylinder value for Young's modulus) by the elastic camber. It is seen by Eq. (9) that this coefficient corresponds to the creep coefficient. Hence the time-dependent camber is a direct result of concrete creep. In fact, the camber coefficient and creep coefficient would be equal except that the camber coefficient includes the effect of prestress loss.

The same solution is obtained in camber calculations if the camber coefficient is included in the expression for the sustained modulus. In this case

$$E_{sus} = \frac{E_p}{1 + B_t} \qquad (17)$$

or

$$\frac{E_{sus}}{E_p} = \frac{1}{1 + B_t} \qquad (18)$$

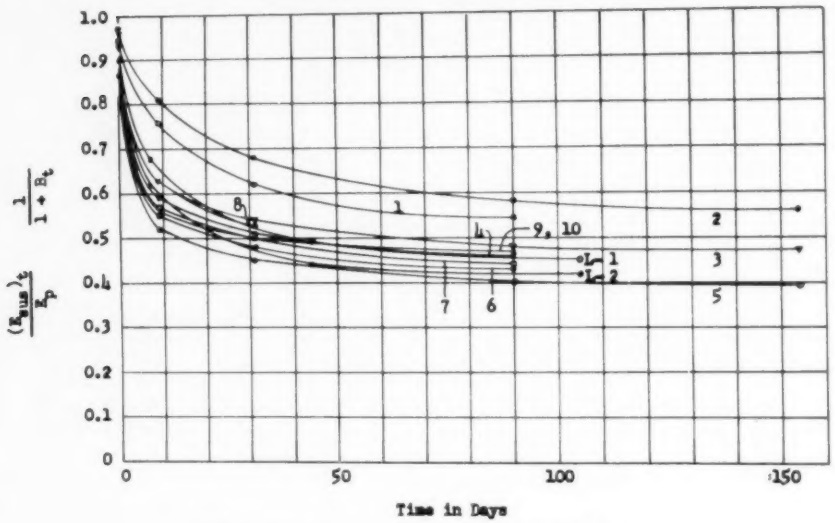
where E_p is the cylinder modulus at the time of prestressing, B_t is the camber coefficient (actually a creep coefficient but including the elastic and inelastic effect of prestress loss), and E_{sus} is the sustained modulus to be used for any time t to determine the total camber. The values for the camber coefficient B_t versus time are shown in Fig. 12 and for E_{sus}/E_p versus time in Fig. 13 for the test beams.

DISCUSSION OF RESULTS

The experimental data included in the camber investigation provided an insight into the nature of initial-plus-time-dependent camber deformation. The midspan camber for each beam was measured directly using dial gages for the noncomposite beams and a level for the composite beams. Measured strains were also used in the camber study.

The equations previously developed for solving the camber problem involved, in every case, an experimentally determined coefficient which was primarily a function of concrete creep. These coefficients (C_{v_t}, S_t, B_t, and E_{sus}) were evaluated for the test beams and are shown in Fig. 9, 11, 12, and 13, respectively. The prestress loss coefficient (k_t) was evaluated for each beam and is presented in Fig. 10.

Each of these five sets of curves provide a means of studying the effects of the variables considered in the test beams. The variables



For Beams 1-6, $E_p = 4.25 \times 10^6$ psi
For Beams 7-10, $E_p = 4.01 \times 10^6$ psi
For Beams L-1 and L-2, $E_p = 4.50 \times 10^6$ psi

Fig. 13—Ratio of sustained modulus, E_{sus} to cylinder modulus at time of prestressing, E_p versus time

included in the camber study were prestress level, concrete strength, and surrounding atmospheric conditions (principally relative humidity). A summary of the testing conditions and variables considered are shown in Table 3, along with the numerical values for the different coefficients at 90 days following prestressing.

By comparing Beams 1,2; 3,4; and 5,6; the creep coefficient C_{v_t} in the prestressed concrete beam is seen to vary in slightly increasing proportion with magnitude of the prestress force. The effect of the 72 and 90 percent average relative humidities (Beams 7 and 9,10) caused a difference of about 9 percent (1.64 and 1.48, respectively) in the creep coefficient. As would be expected, the value is higher in the lower humidity region. By comparing Beams 3,4; 5,6; and 7; the creep coefficient is seen to vary more closely with prestress level (relative to concrete strength) than to just prestress magnitude itself for different concrete strengths. A comparison of Beams 3,4, and 7 shows the creep coefficient to be about 12 percent greater (1.64 and 1.45) for the weaker concrete under the same prestress force.

The percentage prestress loss varied from 22.4 percent to 18.3 percent for the different prestress levels. The smaller prestress level (Beams 1 and 2) showed the largest percentage of loss in prestress. The reason for this is that the effect of creep on prestress loss is approximately

proportional to stress level, but the effect of concrete shrinkage on prestress loss is independent of prestress level. Hence the lower initial prestress force would have the higher percentage loss. The effect of the 72 and 90 percent average relative humidities (Beams 7 and 9,10, respectively) caused a big difference (20.1 and 12.4 percent, repectively) in the prestress loss percentage. The principal reason for this is the greater concrete shrinkage in the lower humidity region, although the effect of concrete creep also contributed somewhat to this difference. By comparing Beams 3,4, and 7, it is seen that the percentage of prestress loss was less in the stronger concrete beams (18.3 and 20.1 percent, respectively). The reason for this was the smaller creep strain in the stronger concrete since the shrinkage strain was about the same for both.

The same comparisons that were made for the creep coefficient C_{v_t} are seen in Table 3 to closely apply to the summation coefficient S_t and the camber coefficient B_t, since these coefficients were essentially creep coefficients that included the effects of prestress loss in the prestressed concrete beams. However, the camber coefficient B_t and the corresponding sustained modulus E_{sus} ($E_{sus} = E_p/1 + B_t$) are direct func-

TABLE 3 — SUMMARY OF TESTING CONDITIONS AND EFFECTS OF VARIABLES IN CAMBER STUDY

Beam No.	Concrete strength 28 days, psi	Stress level percent f'_{ci}	psi	Location	Average relative humidity, percent	Concrete age when prestressed, days
1,2	5030	25	1250	Lab	77	28
3,4	5030	45	2250	Lab	77	28
5,6	5030	65	3250	Lab	77	28
7,8	3760	60	2250	Lab	72	28
9,10	3760	60	2250	Field	90	28

Experimental values at 90 days following prestressing

Beam No.	C_{v_t}	k_t	S_t	B_t	$\frac{E_{sus}}{E_p}$	E_{sus}†
Prestress level						
1,2*	1.28	0.224	0.93	0.79	0.56	2.38
3,4	1.45	0.183	1.15	1.11	0.47	2.00
5,6	1.70	0.184	1.39	1.42	0.42	1.78
Relative humidity						
7	1.64	0.201	1.33	1.25	0.44	1.77
9,10	1.48	0.124	1.28	1.14	0.47	1.89
Concrete strength						
3,4	1.45	0.183	1.15	1.11	0.47	2.00
5,6	1.70	0.184	1.39	1.42	0.42	1.78
7	1.64	0.201	1.33	1.25	0.44	1.77

*Average values were used for the beam pairs except for Beam 8 which was used to form a composite beam at 37 days following prestressing.

†The sustained modulus E_{sus} is in psi $\times 10^6$. The other coefficients are dimensionless quantities.

tions of the initial-plus-time-dependent camber deformation and the effects of the three variables considered on these values are a direct indication of their effects on total camber. The camber coefficient is the percentage increase in camber above the computed initial camber (in decimal form) using the cylinder value for Young's modulus at the time of prestressing, E_p.

The sustained modulus E_{sus} includes the effects of both concrete creep and prestress loss. The values of E_{sus}/E_p (0.56, 0.47, and 0.42) for Beams 1,2; 3,4; and 5,6, respectively, show the effect of prestress level on the sustained modulus. It is seen that the lowest prestress level beam has an increasingly higher value because of the greater percentage prestress loss. The effect of the 72 and 90 percent average relative humidities (Beams 7 and 9,10, respectively) caused a difference of about 7 percent (0.44 and 0.47, respectively) in the values of E_{sus}/E_p and thus a difference in the values of the total camber deformation. The lower humidity region had the lower sustained modulus value and the larger total camber as would be expected.

By comparing Beams 3,4; 5,6; and 9,10 ($E_{sus}/E_p = 0.47$, 0.42, 0.44, and 0.47 or $E_{sus} = 2.00$, 1.78, 1.77, and 1.89×10^6, respectively), it is considered significant that the total camber was noticeably insensitive to the effects of relative humidity and concrete strength. The reason for this is that there is a tendency for the effect of greater creep and resulting greater camber growth to be offset by a greater loss of prestress caused by both shrinkage and creep and the resulting reduction in camber growth. The maximum difference in the total camber (as reflected in the E_{sus}/E_p values) was about 11 percent. This was true even for Beams L-1 and L-2 of Lofroos and Ozell[2] ($E_{sus}/E_p = 0.44$ and $E_{sus} = 1.98 \times 10^6$) which were 5 ft longer and 2 in. wider than the other beams tested.

Troxell[3] determined that the shrinkage and creep of concrete at 90 days are about 60 percent of their ultimate values. Based on this percentage, the ultimate value for the creep coefficient C_{v_t} is 2.3 for the usual prestress levels. This was found to vary only slightly between the laboratory and field conditions of these tests and the different concrete strengths used. The ultimate percentage of prestress loss was found to be about 30 percent for the laboratory conditions and about 20 percent for the field conditions. These values are considerably higher than those used in current design practice (about 15 percent for post-tensioned beams). One reason for using this lower value may be that concrete strength also increases with age. However, for Type I cement, the concrete strength at 90 days is approximately 90 percent of its ultimate value. Hence it is believed that higher values for the percentage of prestress loss than the ordinary rule of thumb value of 15 percent for post-tensioned beams are in order. The methods used

for this part of the experimental investigation were substantiated by two independent types of measurements.

The summation coefficient S_t, the camber coefficient B_t, and the sustained modulus E_{sus} do not follow the same time-dependent increase for long periods. As the curves in Fig. 11, 12, and 13 indicate, the ultimate values appear to have been nearly reached at the end of the test period. The curves for the creep coefficient C_{v_t} (Fig. 9) and the prestress loss coefficient k_t (Fig. 10) are seen to be decidedly steeper or continuing to increase more at the end of the test period. The reason for this will now be discussed in relation to the ultimate camber deformation in prestressed concrete beams.

There is little doubt that concrete shrinkage and creep continue appreciably for long periods. However, the total camber deformation (determined by the sustained modulus) is primarily a function of creep only, since the sustained modulus for a constant sustained stress is given by

$$E_{sus} = \frac{E_c}{1 + C}$$

where C is the creep coefficient. But in the case of the prestressed concrete beam, the sustained prestress force is not constant, and concrete creep also tends to reduce camber growth through its effect on the loss of prestress. In addition, concrete shrinkage has the main function of reducing the camber growth through its effect on the loss of prestress.

It appears evident, therefore, that the camber growth would tend to reach a maximum value relatively early compared to the long-time shrinkage and creep strains that would be expected. In fact, there may even begin to be a reduction in camber or downward deflection after a few hundred days. This would be brought about through the continued loss of prestress resulting from both shrinkage and creep plus any effect that time may have on increasing the frictional force

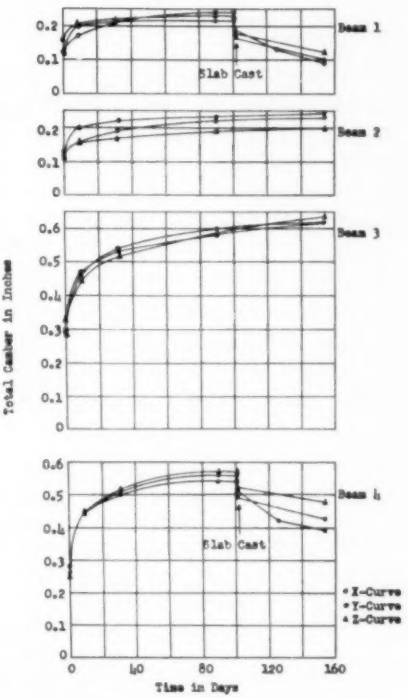


Fig. 14—Comparison of midspan camber by three methods (Beams 1-4)

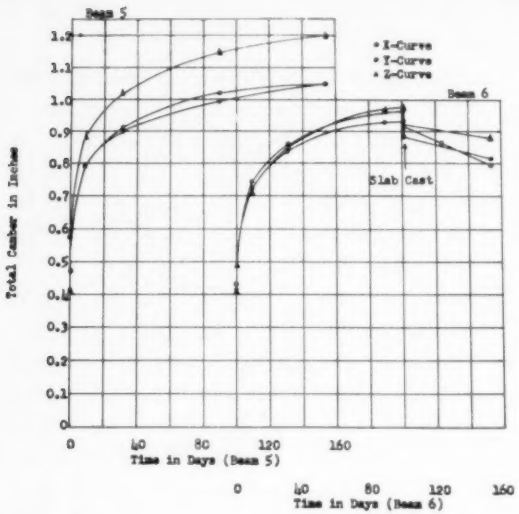


Fig. 15—Comparison of midspan camber by three methods (Beams 5 and 6)

at the roller end. For camber growth to continue freely, the two ends of the beam at the bottom must necessarily be permitted to decrease their distance between each other. The measured camber curves in Fig. 7 show that the total camber for the test beams seem to reach their maximum values from about 100 to 200 days depending on the prestress level.

For ordinary prestress levels, the ultimate values for the respective coefficients appear to be $S_u = 1.4$, $B_u = 1.2$, and $E_{sus}/E_p = 0.45$. The value for the corresponding ultimate sustained modulus to be used in camber calculations is $E_{sus} = 1.9 \times 10^6$ psi. These values were found to vary only slightly between the different concrete strengths and relative humidities of this investigation. It was also found for ordinary prestress levels that the camber increased about 10 percent above the computed initial camber during the first ½ hr, about 30 percent during the first 24 hr, and about 110 percent after 90 days following prestressing. These figures correspond to sustained moduli of $E_p/1.10$, $E_p/1.30$, and $E_p/2.10$, respectively, where E_p is the cylinder value for Young's modulus at the time of prestressing.

The midspan camber for each beam was determined in three independent ways and compared in Fig. 14-16. The three different methods referred to as X, Y, and Z are described as follows:

X—Camber from direct measurement using dial gages for the noncomposite beams and level readings for the composite-beam portion of the curves. This is exactly the same as the camber that could be computed using the camber coefficient B_t or the sustained modulus E_{sus} since these factors were determined from the measured camber.

Y—Computed camber using the basic camber equation involving the summation coefficient S_t (Eq. (8) for the noncomposite beams and Eq. (15) for the composite beams). The parabolic cable solution was necessary because of the displacement of the unbonded bars relative to the centroidal axis during the camber deformation. The bar eccentricities and prestress forces were determined by measurement.

Z—Camber computed by Eq. (20) using the slope of the strain distribution diagrams drawn from the total measured strain less shrinkage strain. A sample of these diagrams is shown in Fig. 6. This was used simply as a third independent determination of the total camber for comparison purposes.

The time-dependent curvature expression for a beam which is bending is given by the expression

$$\frac{d^2y}{dx^2} = -H_x \quad \text{.....(19)}$$

where H_x is the slope of the elastic-plus-creep strain distribution diagram $(\varepsilon/d)_x$ as a function of x. For a parabolic variation of H_x, the midspan camber solution becomes

$$y = (H_{end} + 5\,H_{mid})\,L^2/48 \quad \text{.....(20)}$$

This equation is used in connection with Method Z for calculating camber.

The camber curves in Fig. 14 through 16 were determined for each beam by the three methods X, Y, and Z. The three curves for each beam were found to be in reasonably close agreement, which substantiates the experimental data and the formulas presented for computing camber deformation.

An understanding of the relative influences of the different factors affecting the total camber of composite beams can be obtained by referring to Table 4, which gives an explanation and solution for each term of Eq. (15). There is thought to be no short method for calculating the time-dependent camber in composite beams, since several factors

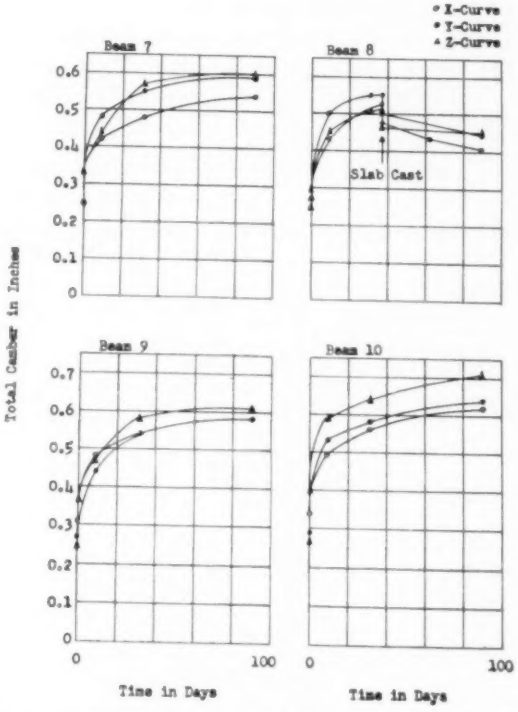


Fig. 16—Comparison of midspan camber by three methods (Beams 7-10)

TABLE 4 — CAMBER SOLUTION FOR COMPOSITE BEAMS

$$y = \left[\underset{(1)}{\frac{1}{E_p I_2}(1 + S_q)\frac{P_i L^2}{48}(e_o + 5e_m)} - \underset{(2)}{\frac{1}{E_p I_2}(1 + C_{v_i}\frac{5wL^4}{384}} - \underset{(3)}{\frac{1}{E_q I_2}\frac{5w'L^4}{384}} \right]^{*}$$

$$+ \left[\underset{(4)}{\frac{S'_{t'}}{E_p I_2}\frac{P_i L^2}{48}(e_o + 5e_m)} - \underset{(5)}{\frac{C'_{v_{t'}}}{E_p I_2}\frac{5wL^4}{384}} - \underset{(6)}{\frac{C_{v_{t'}}}{E_q I_2}\frac{5w'L^4}{384}} \right.$$

$$\left. - \underset{(7)}{\frac{1 + C_{v_{t'}}}{E_q I_2}\tfrac{2}{3}F c_{2T}\frac{L^2}{8}} \right]^{\dagger} \left[\underset{(8)}{\frac{E_2 I_2}{E_1 I_1 + E_2 I_2}} \right] \qquad (15)$$

(1) Elastic-plus-inelastic camber resulting from the prestress force up to the time of slab casting $t = q$

(2) Elastic-plus-inelastic deflection resulting from the precast beam dead load up to the time of slab casting

(3) Elastic deflection resulting from slab dead load

(4) Inelastic camber resulting from the prestress force after the slab was cast or time t' (Note $S_t = S_q + S'_{t'}$ or $S'_{t'} = S_t - S_q$)

(5) Inelastic deflection resulting from the precast beam dead load after the slab was cast (Note $C_{v_t} = C_{v_q} + C'_{v_{t'}}$ or $C'_{v_{t'}} = C_{v_t} - C_{v_q}$)‡

(6) Inelastic deflection resulting from the slab dead load

(7) Total deflection resulting from differential shrinkage

(8) Factor which includes the slab resistance to bending

Beam No.	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1-C	0.392	−0.168	−0.050	0.012	−0.008	−0.036	−0.049	0.950
4-C	0.721	−0.179	−0.050	0.036	−0.013	−0.041	−0.049	0.950
6-C	1.112	−0.185	−0.050	0.048	−0.008	−0.045	−0.049	0.950
8-C	0.738	−0.177	−0.054	0.058	−0.018	−0.060	−0.051	0.948

Beam No.	[(1)+(2)+(3)] = (9)	[(4)+(5)+(6)+(7)](8) = (10)	y = (9)+(10), in.	Time, t, days
1-C	0.176	−0.077	0.099	154
4-C	0.492	−0.064	0.428	154
6-C	0.877	−0.051	0.826	154
8-C	0.507	−0.067	0.440	90

*Total camber immediately after slab casting = [(1) + (2) + (3)] = (9)

†Total camber or deflection for time t' which is the time measured from the day of slab casting = [(4) + (5) + (6) + (7)](8) = (10)

‡Note that the $C_{v_{t'}}$ values were determined as follows:

Beams 1-C, 4-C, and 6-C; 60 percent and for Beam 8-C; 80 percent of the corresponding values when loaded at 28 days were used.

must be considered, as indicated by Eq. (15). This equation is helpful in understanding the relative importance of the different factors and could be used without too much difficulty in design. The values for the differential shrinkage force F used in the solutions shown in Table 4 were taken from Reference 11.

CONCLUSIONS

The following conclusions are drawn concerning the time-dependent effects in both noncomposite and composite post-tensioned concrete members:

1. For design purposes, the total (initial-plus-time-dependent) camber deformation for noncomposite beams is readily obtained by Eq. (10) using either the camber coefficient B_t or the sustained modulus E_{sus}. However, the basic camber equation developed herein, Eq. (5), is required for a more rational analysis of noncomposite beam camber. The rational analysis of composite beam camber requires the solution of Eq. (15) or (16). These two equations are not practical for design purposes. However, any simpler method is not recommended since it would require the use of approximations that may lead to erroneous results.

2. Camber deformation for ordinary prestress levels ($0.45 f'_{ci}$) increases about 10 percent above the computed initial camber (using the cylinder modulus of elasticity) during the first ½ hr, about 30 percent during the first 24 hr, and about 110 percent during the first 90 days following prestressing.

3. Total camber deformation is noticeably insensitive to the effects of relative humidity and concrete strength, because there is a tendency for greater time-dependent concrete strains resulting from creep and shrinkage to be offset by greater losses of prestress. The maximum difference in the total camber of the test beams was about 11 percent for these two variables.

4. Total camber deformation appears to be more closely related to the prestress level (relative to concrete strength) than to the actual prestress magnitude itself.

5. Total camber deformation may reach its ultimate value relatively early (100 to 200 days) compared to the long-time shrinkage and creep strains that would be expected. The following ultimate values are recommended for ordinary prestress levels in post-tensioned members: the prestress loss percentage of 20 percent for average relative humidity of about 90 percent and 30 percent for average relative humidity of about 70 percent, the creep coefficient $C_{v_u} = 2.3$, the summation coefficient $S_u = 1.4$, the camber coefficient $B_u = 1.2$, and the sustained modulus $E_{sus}/E_p = 0.45$ or $E_{sus} = 1.9 \times 10^6$ psi. These values for C_{v_u}, S_u,

B_u, E_{sus}/E_p, and E_{sus} are subject to some variation for different concrete strengths and relative humidities, but it is believed that for high-strength concrete the variation would be relatively small.

ACKNOWLEDGMENTS

The authors wish to express their appreciation to the Florida State Road Department for sponsoring the research program of which this investigation was a part. Acknowledgment is also made to Stressteel Corp. and Flexico Products, Inc. for the donation of the post-tensioning bars and flexible tubing used in the preparation of the test specimens.

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Title No. 57-69

Durability of Concrete in Sea Water

By INGE LYSE

Paper deals with the durability of concrete when exposed to freezing and thawing in sea water. Extensive experimental investigations have for the past 20 years been carried out at the concrete laboratory of Norway's Institute of Technology, Trondheim, and the most important results from these investigations are reported here.

Among the more important results is that freezing and thawing in sea water is much more detrimental to the durability of the concrete than is the freezing and thawing in fresh water. Furthermore, it is shown that the amount of entrained air necessary for giving the highest resistance of the concrete to frost action in sea water is in the range of 10 to 12 percent, which is more than twice as large as for concrete in fresh water.

■ In a country with considerable frost during the winter season, concrete is exposed to the detrimental effect of frost action. Although a concrete which is more or less dry during the frost period will not be specially vulnerable to freezing and thawing, a concrete saturated with water will be subjected to the bursting action as the water in the concrete freezes and the subsequent release of this internal pressure as the ice melts during thawing. The action of freezing and thawing is thus a repetition of loadings and unloadings of the internal structure of the concrete. The rapidity at which freezing and thawing takes place and the number of these cycles are of considerable importance for the durability of concrete.

The materials which make up concrete as well as the proportioning of the materials have an important effect on the resistance of concrete from frost action. But, in addition, the length of time of effective curing of the concrete before it is exposed, plays an important role. Furthermore, the amount of the entrained air voids in the concrete has been found to be a major factor for the durability of concrete. In concrete exposed to freezing and thawing in fresh water the amount of entrained air voids varies generally between 3 and 5 percent of the volume of the concrete, but for concrete in sea water the amount of air voids must be raised to between 10 and 12 percent for the best resistance. While concrete exposed to frost in fresh water has been widely studied, concrete in sea water has only recently been included in durability studies.

An ACI member since 1926, **Inge Lyse** is professor and director of the concrete laboratory, Norway's Institute of Technology, Trondheim. Dr. Lyse is a past president of RILEM, the Norwegian Concrete Institute, and Norway's Academy of Technical Sciences. He was an ACI Director (1937-1938) and is currently a member of ACI Committees 115, Research, and 201, Durability of Concrete in Service.

Because of this the results included in this paper are limited to concrete exposed to freezing and thawing in sea water.

In Norway the problem of the disintegration of concrete in sea water has long ago been realized in specific requirements for reinforced concrete structures in harbor constructions. The corrosion of the reinforcing steel in harbor structure has lead to the requirement for the thickness of the concrete covering of the reinforcement of at least 2 in., and for concrete cast under water this has been increased to about 3 in. Furthermore, it is generally accepted that the area of the concrete between the water level at high tide and at low tide shall be protected using 2 in. thick wood planks, well-impregnated and maintained in place with copper wire or galvanized steel wire. This protective cover which amounts to making the formwork in this way and leaving it permanently on the structure, has been found to give excellent protection against concrete disintegration. As previously mentioned[1] the protective cover acts as an insulator in such a way that at moderate freezing temperatures of the ambient atmosphere the frost will not penetrate the 2 in. wood planks during the few hours of low tide so that the concrete remains unfrozen all the time. At low temperatures the frost will penetrate the covering and the concrete behind the planks will also remain frozen during high tide. The protective cover will thus reduce appreciably the cycles of freezing and thawing during the winter.

EXPERIMENTAL RESULTS

The durability tests have been carried out in a refrigeration room of the concrete laboratory. The description of the refrigeration room and the details of the method of tests used are given in an earlier paper.[2] In the tests reported here the specimens (4-in. cubes) were submerged in sea water during the thawing period and placed directly in a rack for freezing in the refrigeration room. As the emphasis in this paper is on the durability in sea water, the fresh water results are omitted except for a direct comparison between fresh water and sea water as a thawing medium.

The results of the comparison between thawing in fresh water* and in sea water† are presented in Fig. 1. It is seen here how much more severe is sea water than fresh water on the durability of concrete. These results are for two different mixes of concrete, $C/W = 1.5$ and

*The fresh water was ordinary drinking water in the city of Trondheim.
†The sea water had an average salt content of 3.5 percent.

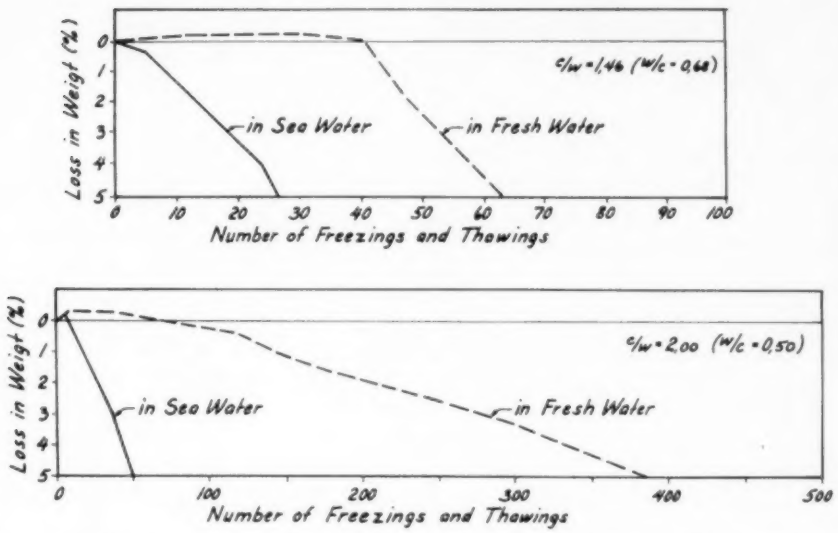
Loss in Weight (%)
C/w = 1.46 (w/c = 0.68)
in Sea Water
in Fresh Water
Number of Freezings and Thawings
C/w = 2.00 (w/c = 0.50)
in Sea Water
in Fresh Water
Number of Freezings and Thawings

Fig. 1—Effect of thawing in fresh water and in sea water on the durability of concrete

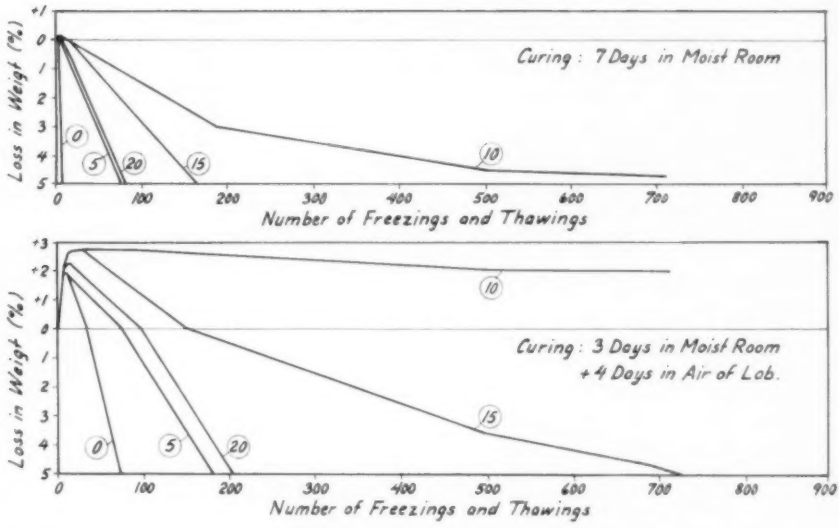

Fig. 2—Durability results of concrete for $C/W = 1.50$ ($W/C = 0.67$) in sea water; age of specimens at beginning of test: 7 days; number in circles indicates theoretical percentage of air voids in concrete

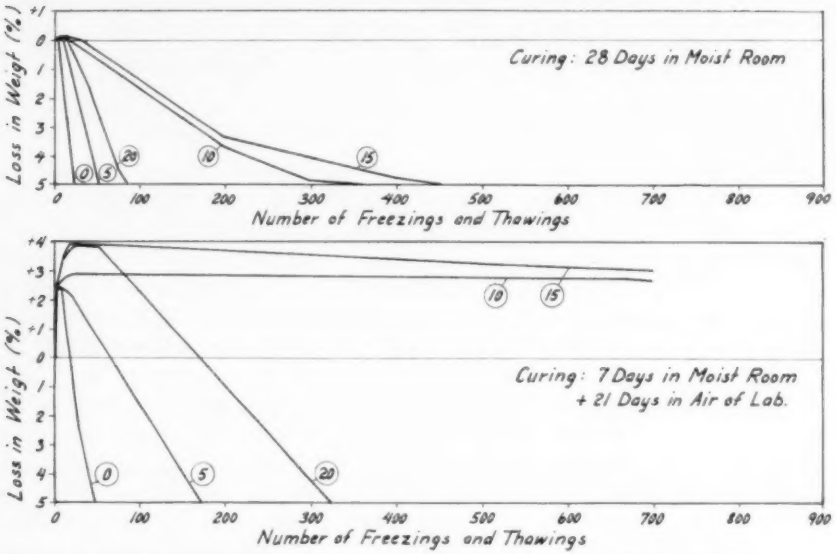
Loss in Weight (%)
Curing: 28 Days in Moist Room
Number of Freezings and Thawings
Curing: 7 Days in Moist Room
+ 21 Days in Air of Lab.

Fig. 3—Durability results of concrete for $C/W = 1.50$ ($W/C = 0.67$) in sea water; age of specimens at beginning of test: 28 days; number in circles indicates theoretical percentage of air voids in concrete

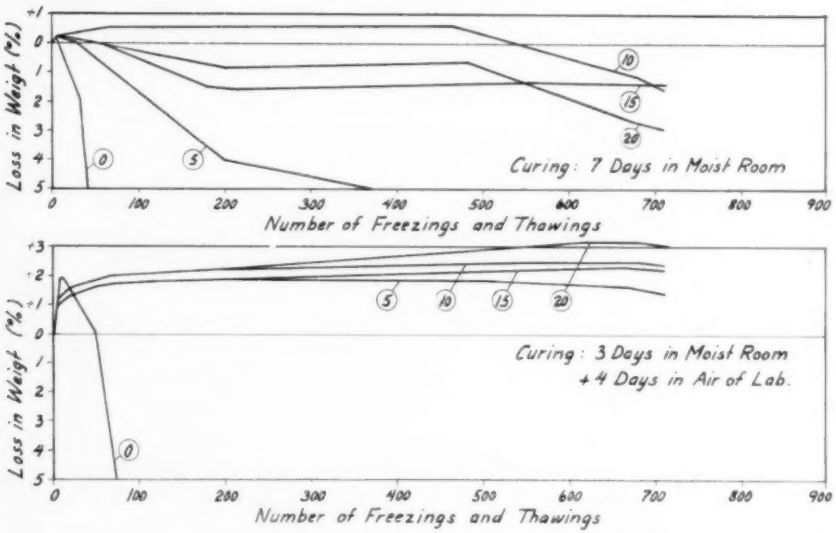

Fig. 4—Durability results of concrete for $C/W = 2.00$ ($W/C = 0.50$) in sea water; age of specimens at beginning of test: 7 days; number in circles indicates theoretical percentage of air voids in concrete

$C/W = 2.0$ by weight ($W/C = 0.67$ and 0.50), without any entrained air. For air-entrained concrete the difference appears still greater. For this reason sea water has been accepted for our durability test of concrete as a test that can be carried out in a much shorter time than in fresh water.

The most important durability factors investigated were effect of:

1. Size and gradation of the aggregates
2. C/W ratio of the concrete
3. Amount of entrained air in the concrete
4. Length of moist curing prior to the durability tests

In studying the effect of the aggregates special emphasis was placed on the maximum size of the aggregates (which was varied from No. 4 to 1½ in.) and the amount of fine material passing the No. 200 sieve. The results showed that the maximum size of the aggregates as well as the amounts of fine material in the sand had relatively small effect on the durability of concrete in sea water.

The effect of the C/W ratio* on the durability of the concrete is shown in Fig. 2-7. Although there is only a small effect of the C/W

*The C/W ratios by weight were 1.50, 2.00, and 2.50. The consistency of the mix was maintained between 3 and 4 in. This gave an average mix by weight of:
1: 3.5: 3.5 for $C/W = 1.50$
1: 2.4: 2.4 for $C/W = 2.00$
1:1.65:1:65 for $C/W = 2.50$
Natural sand and crushed gravel were used as aggregates.

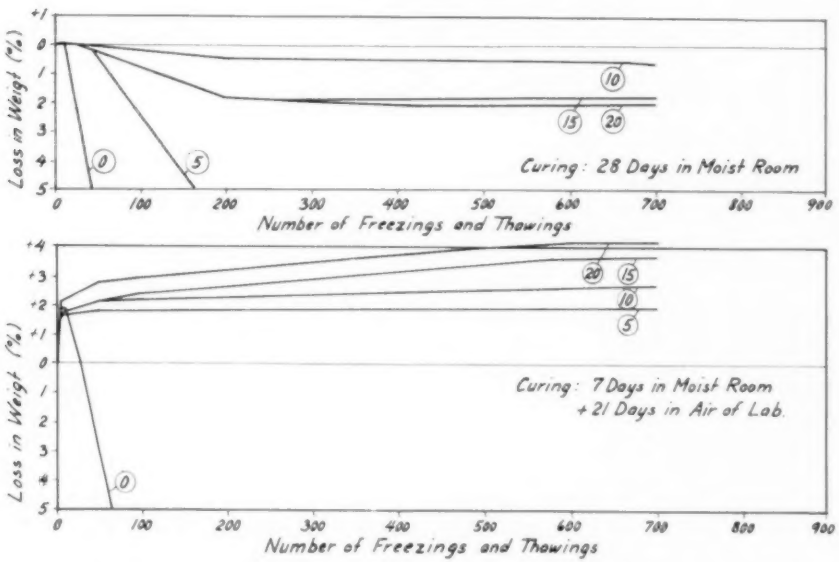
Loss in Weight (%)
Curing: 28 Days in Moist Room
Number of Freezings and Thawings
Curing: 7 Days in Moist Room
+21 Days in Air of Lab.
Number of Freezings and Thawings

Fig. 5—Durability results of concrete for $C/W = 2.00$ ($W/C = 0.50$) in sea water; age of specimens at beginning of test: 28 days; number in circles indicates theoretical percentage of air voids in concrete

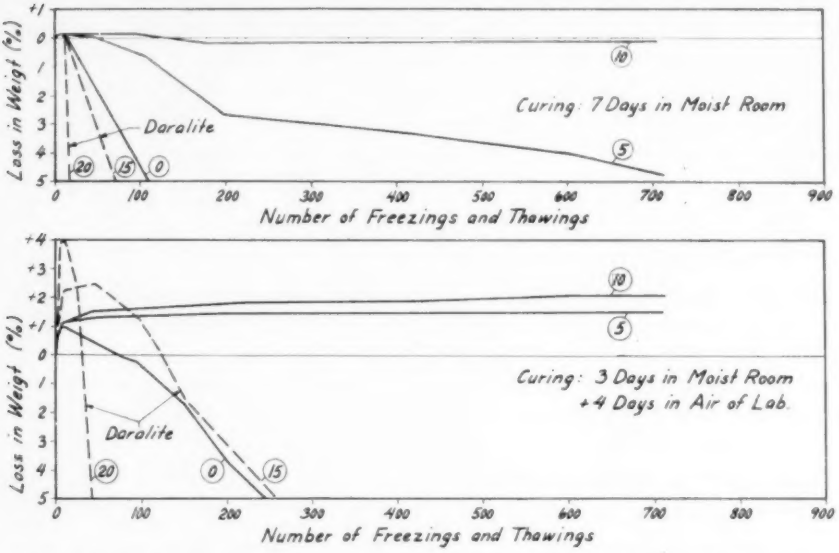

Fig. 6—Durability results of concrete for $C/W = 2.50$ $(W/C = 0.40)$ in sea water; age of specimens at beginning of test: 7 days; number in circles indicates theoretical percentage of air voids in concrete

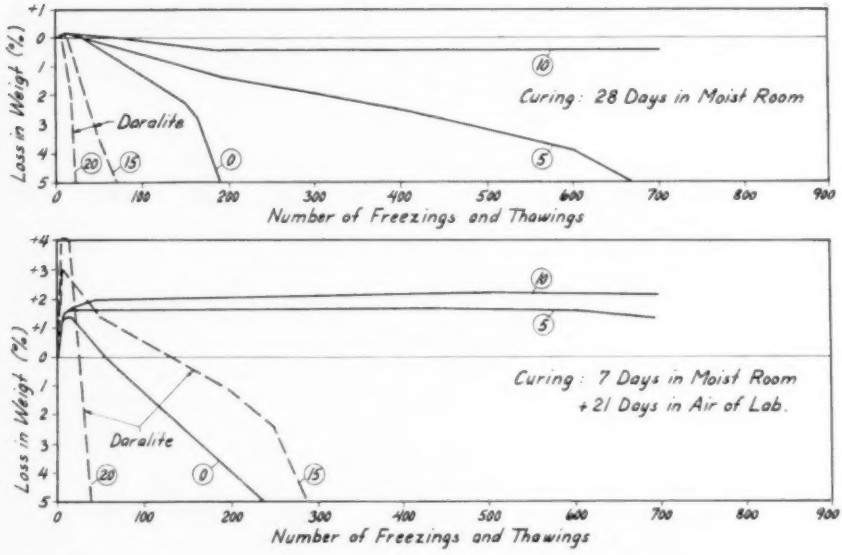

Fig. 7—Durability results of concrete for $C/W = 2.50$ $(W/C = 0.40)$ in sea water; age of specimens at beginning of test: 28 days; number in circles indicates theoretical percentage of air voids in concrete

ratio on the durability of concrete in sea water when no air entrainment is used, a marked effect is apparent as soon as the amount of entrained air in the concrete becomes important. With a sufficient amount of entrained air in the concrete the effect of the C/W ratio on the frost resistance in sea water becomes just about as important as the C/W ratio is for the frost resistance in fresh water.

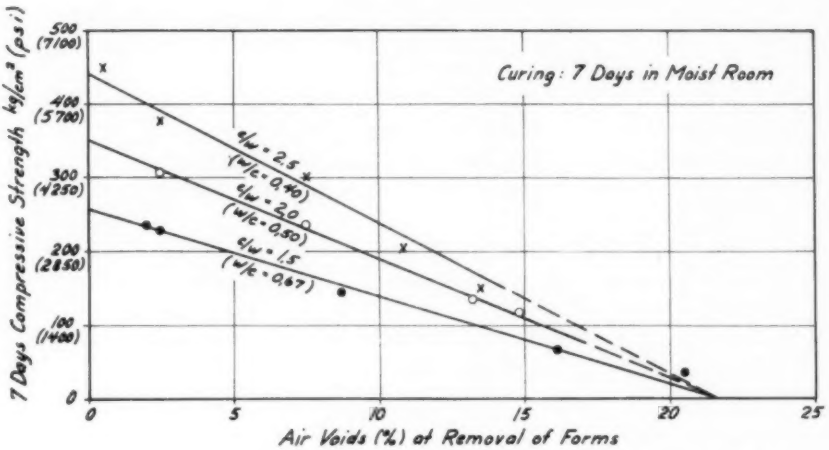

Fig. 8—Compressive strength of concrete of various percentages of air voids; 7 days moist curing

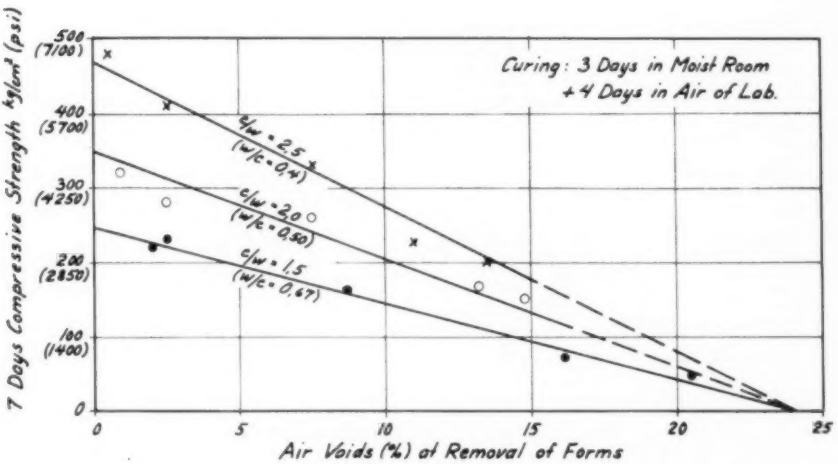

Fig. 9—Compressive strength of concrete of various percentages of air voids; 3 days moist plus 4 days air curing

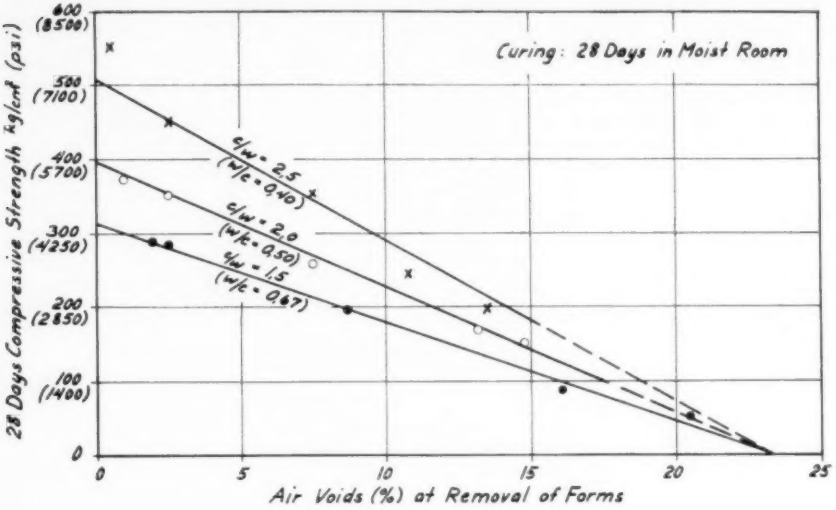

Fig. 10—Compressive strength of concrete of various percentages of air voids; 28 days moist curing

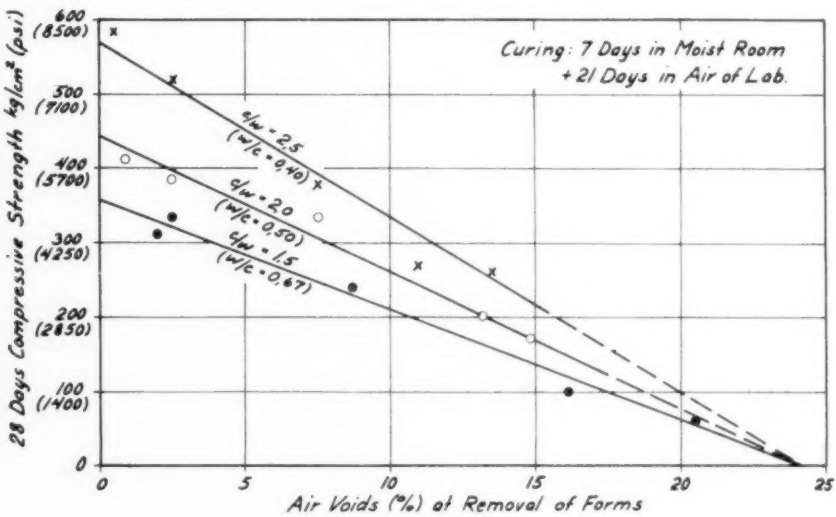

Fig. 11—Compressive strength of concrete of various percentages of air voids; 7 days moist plus 21 days air curing

The moist curing period of the concrete has been found to be of significant importance for the frost resistance in fresh water. However, in sea water the moist curing period has only a small effect as shown in Fig. 2-7. However, the amount of drying out of the concrete before it is exposed to the freezing and thawing test is considerable as can be seen in Fig. 2-7. This indicates that concrete with C/W ratio of 1.5 or more is so impermeable that it does not become fully saturated on being submerged in sea water during the thawing periods. To obtain as much as 15 and 20 percent entrained air in the concrete of $C/W = 2.5$ it was necessary to use a special air-entraining agent (Daralite). This agent did not give stable air voids and gave results indicating special detrimental effect on the frost resistance of the concrete as compared to the air-entraining agent used in all other mixes. As seen from Fig. 8-11 the measured air voids were considerably lower than the theoretical values.

The effect of the entrained air on the compressive strength of the concrete (4-in. cubes) is shown in Fig. 8-11. It is noted that for concrete having 10 to 12 percent entrained air the strength was reduced to about one-half of the strength without air-entrainment.

CONCLUSIONS

From the results obtained to date it appears safe to draw the following conclusions regarding the durability of concrete exposed to freezing and thawing in sea water:

1. The effect of freezing and thawing of concrete in sea water is much more severe than in fresh water.

2. The maximum size of the aggregates and the amount of fines (less than the No. 200 sieve) in the aggregate are of minor importance for the frost resistance of concrete in sea water.

3. The length of time of moist curing before the exposure to freezing and thawing has only a slight effect on the resistance of the concrete to frost action in sea water.

4. A drying out of the concrete prior to its exposure to freezing and thawing in sea water will increase the durability of the concrete in a marked degree.

5. The most important factor for the durability of concrete in sea water is the C/W ratio combined with a large percentage of entrained air (10 to 12 percent).

6. While for resistance to frost action in fresh water the percentages of entrained air for best results ordinarily range from 3 to 6 percent, the percentages giving best durability results for concrete in sea water range from 10 to 12 percent even though this reduces the compressive

strength of the concrete to about one-half of the strength without air-entrainment.

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CONCRETE BRIEFS

Notes from Field and Office

Effect of Geometry in the Economical Design of Cylindrical Shells

By A. M. OZELL* and A. L. CONYERS†

THE PAPER by James Chinn,‡ and the contribution by A. L. Parme and H. W. Conner in their discussion of the same paper have been helpful to the design engineer in understanding the structural behavior of cylindrical shell structures and the simplified method of their design and analysis.

In the design of cylindrical shell structures, the engineer is confronted with the problem of choosing the geometry of the shell which would ensure the most economical shell for a given span. If the span is predetermined by architectural considerations and layout, then the variables which remain to be resolved are: ϕ_k, r, or rise to span ratio, where ϕ_k is the angle from vertical to edge of shell and r is the radius of the shell.

Of all the bending moments, and normal and shearing forces that exist throughout the shell, the most important values that govern the design (i.e., thickness of the shell and the amount of reinforcement) are T_x (longitudinal stress) and M_ϕ (moment acting on the arch) at the crown and at the edge.

To aid the designer in the proper choice of shell geometry, curves are presented herein which were prepared for the following variables within the limits indicated: L = 50, 75, 100, 150, and 200 ft; ϕ_k = 30, 45, and 60 deg; h/L (rise to span ratio) = 1/10 and 1/15. An IBM-650 computer was used for the calculations. Dead load of 50 lb per sq ft and a uniformly distributed live load of 25 lb per sq ft were used throughout.

The curves are based on the approximate method given in Chinn's paper. Where corrections taking into account the effect of horizontal displacement due to a uniform load placed on the shell resulted in significant variance in the values of M_ϕ and T_ϕ, the curves so corrected are shown as dotted lines in Fig. 2, 3, 5, and 6. It should be noted, however, that although the corrections for horizontal displacement affect the numerical values considerably they would have little influence on the over-all design of the shell structure, since, for cases considered in this study, magnitudes of moment

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†Instructor of Civil Engineering, University of Florida, Gainesville, Fla.

‡"Cylindrical Shell Analysis Simplified by Beam Method," ACI JOURNAL, V. 30, No. 11, May 1959 (*Proceedings* V. 55), p. 1183; Discussion, V. 31, No. 3, Sept. 1959 (*Proceedings* V. 55), p. 1583.

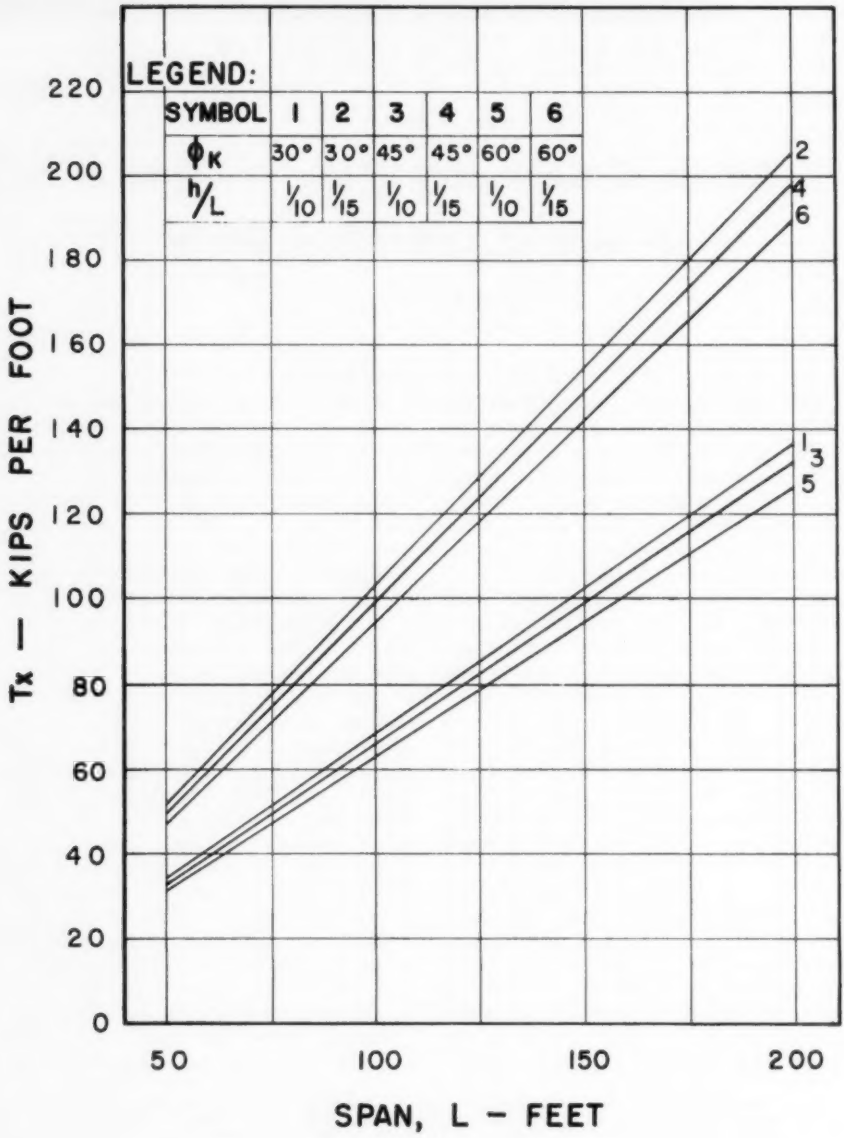

Fig. 1—Variation in T_x at the edge

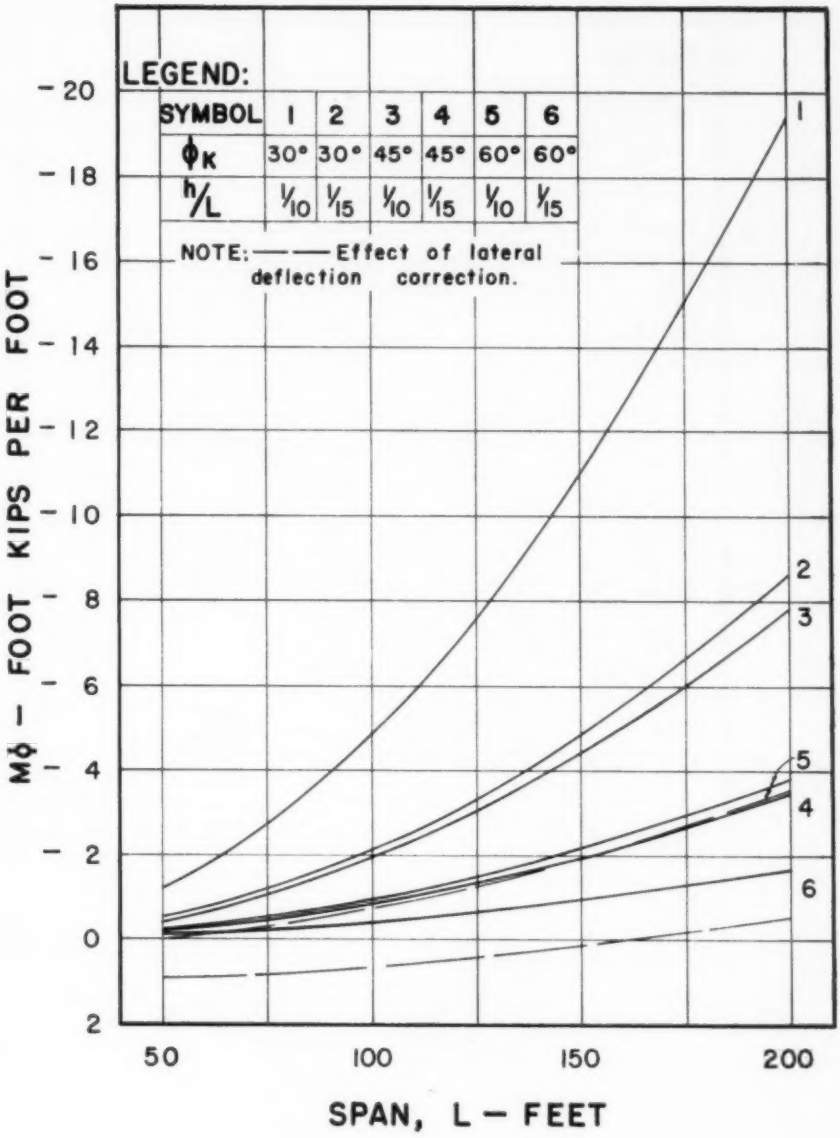

Fig. 2—Variation in M_ϕ at the edge

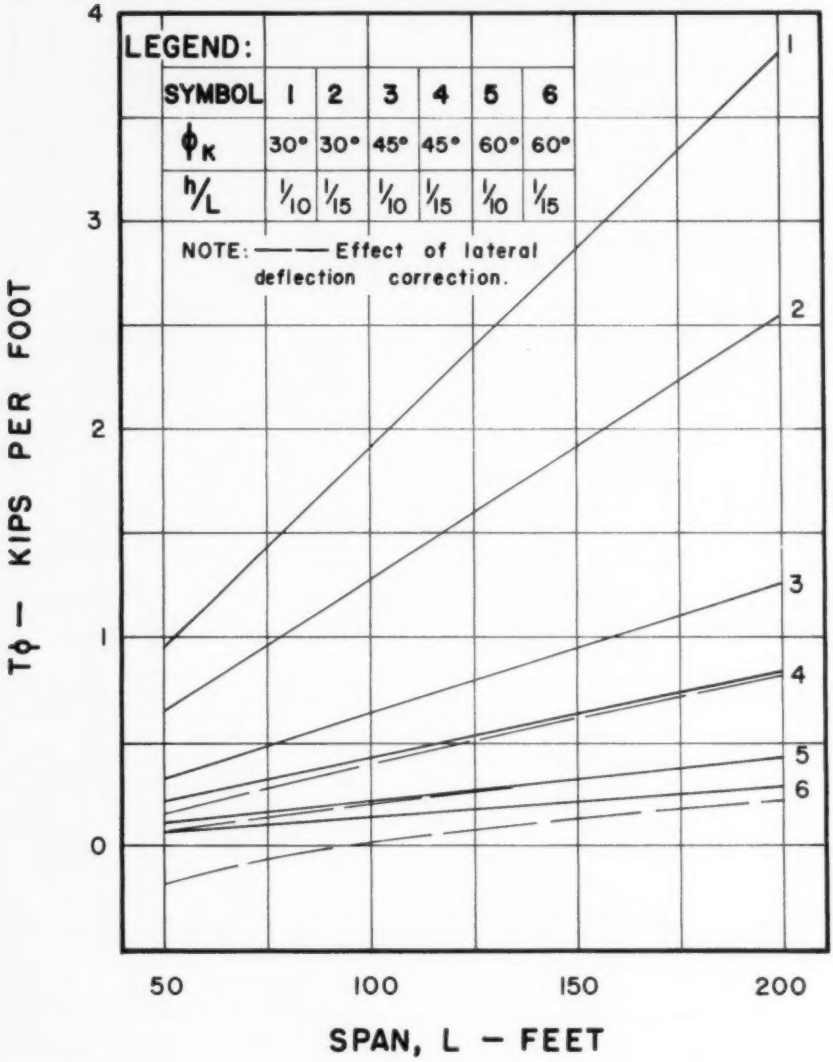

Fig. 3—Variation in T_ϕ at the edge

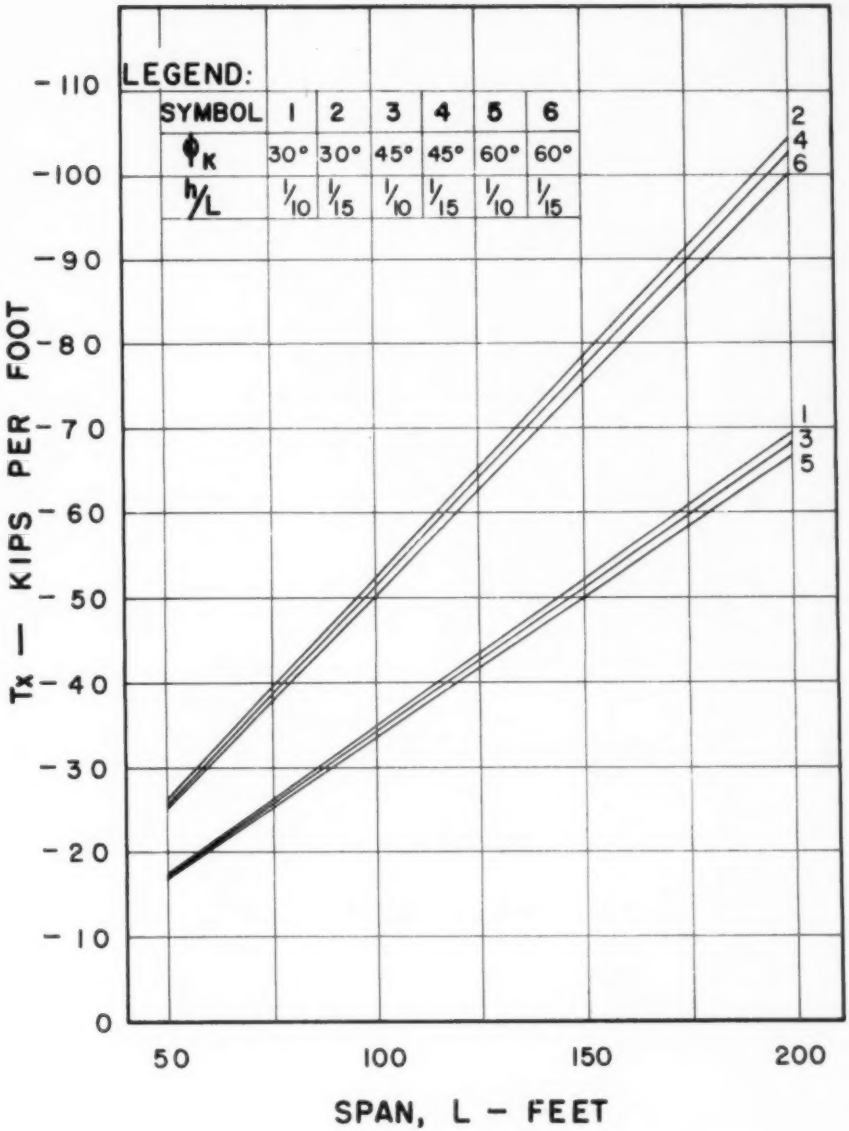

Fig. 4—Variation in T_x at the crown

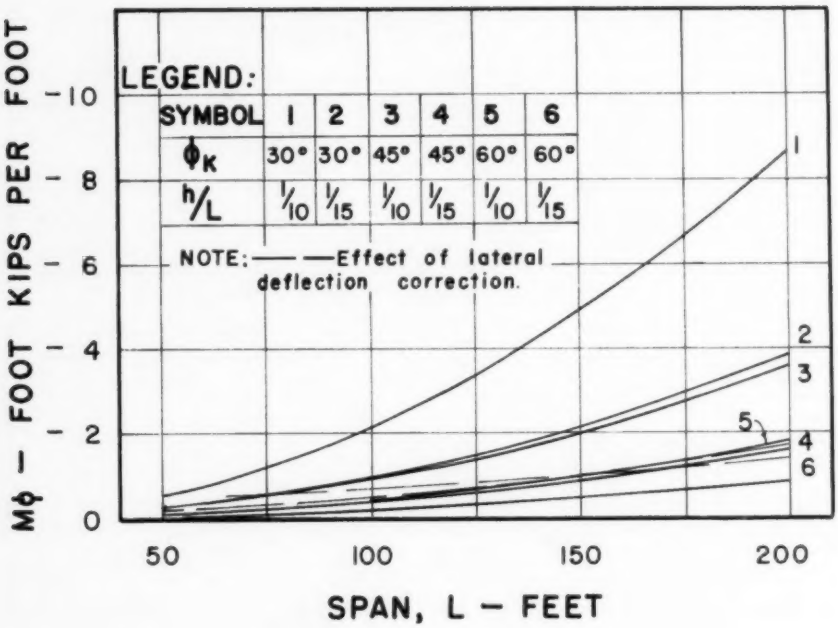

Fig. 5—Variation in M_ϕ at the crown

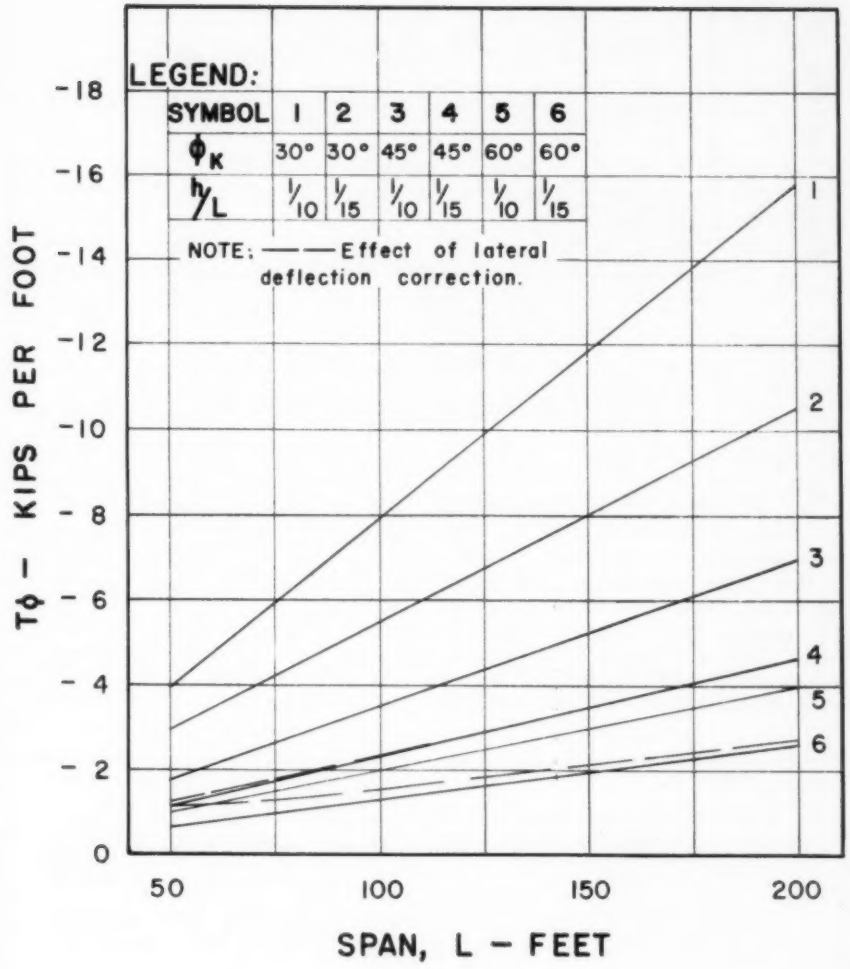

Fig. 6—Variation in T_ϕ at the crown

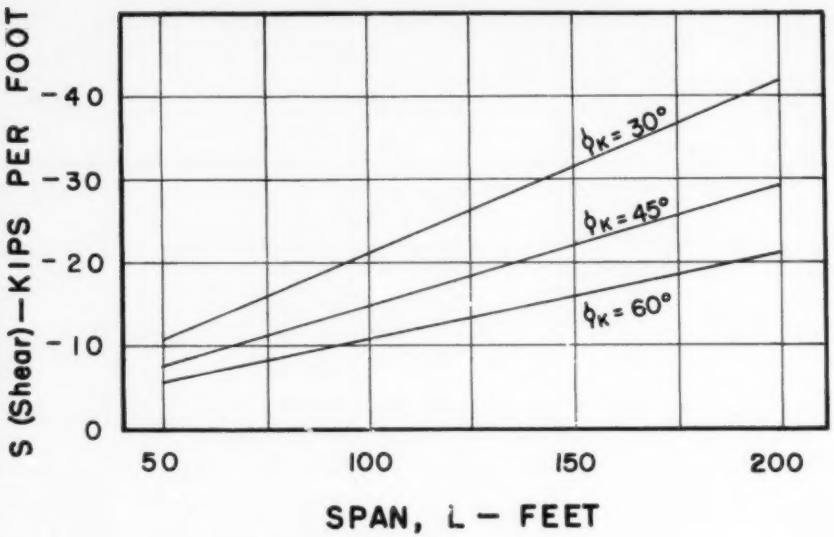

Fig. 7—Variation in maximum shear stress

and thrust so revised are numerically small.

The usefulness of these curves is primarily in the choice of shell geometry for optimum economy. A study of the figures will show that for spans up to 100 ft the relative values of T_s, T_ϕ, and M_ϕ are of the same approximate order of magnitude for angles ϕ_k between 30 to 60 deg and h/L values of 1/10 to 1/15. Hence little gain in economy would result if the most efficient geometry is chosen; although h/L of 1/10 obviously gives the most economical design geometry compared to h/L of 1/15. On the other hand, for spans between 100 to 200 ft, the curves clearly indicate that h/L of 1/10 is by far the better choice for an economical design with ϕ_k of 60 deg giving the most efficient and ϕ_k of 30 deg the least efficient shell geometry.

The Problems and Practices section is an open forum on everyday topics. JOURNAL readers are invited to contribute their solutions to problems that occur in the field or office—whether common or unusual.

Perhaps your solution is a better way of handling some everyday problem that others are looking for. Give your fellow ACI members the benefit of your experience on these problems.

Items appearing in this section can vary from 100 to 1000 words. It is an ideal place for those practical items generally considered too short to be submitted for publication, but containing material of potential importance to a great many.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Periodicals Regularly Scanned For Reviews

The many requests for information concerning the source material of reviews appearing in the Current Reviews section indicate the desirability of publishing a list of those periodicals, bulletins, and proceedings being *scanned regularly* for articles dealing with the field of concrete. If all miscellaneous magazines, bulletins, reports, and books were included (which are reviewed only as they come to the attention of the voluntary reviewers or editors) the following list would cover well over 200 periodicals and/or publishing, educational and research organizations. The following list includes publishers' addresses for convenience should correspondence with them be desired, since copies of articles or books reviewed in the JOURNAL are *not available* through ACI. In most cases they can be obtained directly from the original publishers.

American Ceramic Society Bulletin—American Ceramic Society, 4055 North High St., Columbus 14, Ohio

American Society of Civil Engineers, Proceedings — American Society of Civil Engineers, 33 West 39th St., New York 18, N.Y.

ASTM Bulletin—American Society for Testing Materials, 1916 Race St., Philadelphia 3, Pa.

American Society for Testing Materials, Proceedings — American Society for Testing Materials, 1916 Race St., Philadelphia 3, Pa.

Annales, Institut Technique du Bâtiment et des Travaux Publics—L'Institut Technique du Bâtiment et des Travaux Publics, 6 Rue Paul Valéry, Paris 16e, France

Annales des Ponts et Chaussées—Commission des Annales a l'Ecole Nationale des Ponts et Chaussées, 28 Rue des Saints-Peres, Paris 7e, France

Applied Mechanics Reviews—American Society of Mechanical Engineers, 29 West 39th St., New York 18, N.Y.

Architectural Forum — 9 Rockefeller Plaza, New York 20, N.Y.

Architectural Record — 119 West 40th St., New York 18, N.Y.

Archiwum Inzynierii Ladowej — Kultury i Nauki, Pokoj 2319, Warsaw, Poland

Australian Journal of Applied Sciences —Commonwealth Scientific and Industrial Research Organization, 314 Albert St., E. Melbourne, C 2, Victoria, Australia.

A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 12, June 1961 *Proceedings* V. 57. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are *not* available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

For those members who cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

Battelle Technical Review — Battelle Memorial Institute, 505 King Ave., Columbus 1, Ohio

Bauingenieur, Der—Heidelberger Platz 3, Berlin, Wilmersdorf, Germany

Bauplanung-Bautechnik — Unter den Linden 12, Berlin W8, Germany

Bauzeitung, Die — Hospitalstrasse 12, Stuttgart N, Germany

Béton Armé—22 Rue de L'Arcade, Paris 8e, France

Beton Herstellung und Verwendung — Beton-Verlag GmbH. Taubenstr. 2, Dusseldorf, Germany

Betongen Idag—Norsk Cementforening, Gaustadalleen 30, Oslo-Blindern, Norway

Beton i Zhelezobeton—Moscow, Russia

Betonstein-Zeitung — Kleine Wilhelmstrasse 7, Wiesbaden, Germany

Beton-Teknik — Christians Brygge 28, Copenhagen V, Denmark

Beton- und Stahlbetonbau — Hohenzollerndamm 169, Berlin-Wilmersdorf, Germany

Building Construction Illustrated — 5 South Wabash Ave., Chicago 3, Ill.

BRI Abstracts of Building Science Publications — Building Research Institute, 2101 Constitution Ave., Washington 25, D.C.

Building Research Station Digest — Building Research Station, Garston, Watford, Herts, England

Builleten Stroitelnoi Tekhniki — Moscow, Russia

Bulletin de la Reunion Internationale des Laboratories d'Essais et de Recherches sur les Materiaux et les Constructions — Reunion Internationale des Laboratories d'Essais et de Recherches sur les Materiaux et let Constructions (RILEM), 12 Rue Brancion, Paris 15e, France

Bulletin of Building Technique — Sadovaya 18, Leningrad, Russia

Cahiers du Centre Scientifique et Technique du Batiment — Centre Scientifique et Technique du Batiment, 4 Avenue du Recteur-Poincaré, Paris 16e, France

Cement — Heerengracht 507, Amsterdam C, Netherlands

Cement and Lime Manufacture — 14 Dartmouth St., London, SW 1, England

Cement och Betong — Kolga-huset, Postgiro 1808, Malmo, Sweden

Cemento, Il — Viale Lunigiana, 16 Milano 510, Italy

Centre d'Etudes et de Recherches de l'Industrie des Liants Hydrauliques, Bulletin — Centre d'Etudes et de Recherches de l'Industrie des Liants Hydrauliques, 197 Boulevard Saint-Germain, Paris 7e, France

Ceramic Abstracts — (see *Journal of the American Ceramic Society*)

Civil Engineering — American Society of Civil Engineers, 33 West 39th St., New York 18, N.Y.

Civil Engineering and Public Works Review —8 Buckingham St., London, WC 2, England

Concrete — 400 W. Madison St., Chicago 6, Ill.

Concrete and Constructional Engineering — 14 Dartmouth St., London, SW 1, England

Concrete Construction Magazine — P.O. Box 444, Elmhurst, Ill.

Concrete Products — 79 W. Monroe St., Chicago 3, Ill.

Construccion—Ingenieria Internacional Construccion, 330 West 42nd St., New York, N.Y.

Construction — 92 Rue Bonaparte, Paris 6e, France

Construction Equipment — 205 East 42nd St., New York 17, N.Y.

Constructional Review — 14 Spring St., Sydney, N.S.W., Australia

Consulting Engineer — 227 Wayne St., St. Joseph, Mich.

Contractors and Engineers—470 Fourth Ave., New York 16, N.Y.

Crushed Stone Journal, The—National Crushed Stone Association, 1415 Elliot Place, N.W., Washington 7, D.C.

Danish National Institute of Building Research, Building Research Reports —Danish National Institute of Building Research, 20 Borgergade, Copenhagen K, Denmark

Deutsche Bauzeitschrift — Eickhofstrasse 14/16, Gütersloh, Germany

Deutscher Ausschuss fur Stahlbeton, Proceedings — Bundesallee 215/218, Room 302, Berlin, W 15, Germany

Deutscher Beton-Verein, Proceedings— Deutscher Beton-Verein E. V., Bahnhofstrasse 61, Postfach 627, Wiesbaden, Germany

Engineer, The — 28 Essex St., Strand, London, WC 2, England

Engineering—35-36 Bedford St., Strand London, WC 2, England

Engineering Journal, The — 2050 Mansfield St., Montreal 2, Quebec, Canada

Engineering News-Record — 330 West 42nd St., New York 36, N.Y.

Genie Civil, Le — Rue Jules Lefebvre, Paris 9e, France

Giornale del Genio Civile—Via Nomentana 2, Rome, Italy

Gradbeni Vestnik — Erjavceva lla, Ljubljana, Yugoslavia

Highway Research Abstracts — Highway Research Board, 2101 Constitution Ave., Washington 25, D.C.

Highway Research Board, Bulletin — Highway Research Board, 2101 Constitution Ave., Washington 25, D.C.

Highway Research Board, Proceedings — Highway Research Board, 2101 Constitution Ave., Washington 25, D.C.

Hungarian Technical Abstracts — Hungarian Central Technical Library, P.O. Box 12, Budapest 8, Hungary

In the Field of Building — Building Research Station, Technion — Israel Institute of Technology, Haifa, Israel

Indian Concrete Journal, The — 121 Queen's Road, Bombay 1, India

Industrial and Engineering Chemistry — American Chemical Society, 1155 Sixteenth St., NW, Washington 6, D.C.

Informes de la Construccion — Institute Técnico de la Construccion y del Cemento del Patronato Juan de la Cierva de Investigacciôn Technica, Apartado de Correos No. 2, Costillares, Chamartin, Spain, and Apartado de Correos No. 19,002, Madrid, Spain

Ingenieria—Escuela Nacional de Ingenieria, Ciudad Univistaria, Mexico, D. F., Mexico

Ingenieria Civil — Association de Ingenieros Civiles del Perú, Colmena 788, 40 Piso, Lima, Peru

Ingeniφren — Dansk Ingeniorforening, Ingeniorhuset, 31 V. Farimagsgade, Copenhagen V, Denmark

Institution of Civil Engineers, Proceedings — Institution of Civil Engineers, Great George St., Westminster, London SW 1, England

Institution of Civil Engineers of Ireland, Transactions — Institution of Civil Engineers of Ireland, 35 Dawson St., Dublin, Ireland

Instituto Técnico de la Construccion y del Cemento, Bulletin — Instituto Técnico de la Construccion y del Cemento del Patronato Juan de la Cierva de Investigacion Tecnica, Apartado de Correos No. 2, Costillares, Chamartin, Spain, and Apartado de Correos No. 19,002, Madrid, Spain

Interbuild — 11 Manchester Square, London, W 1, England

International Civil Engineer — Lennox House, Norfold St., London, WC 2, England

Iowa Engineering Experiment Station, Bulletin — Iowa Engineering Experiment Station, Iowa State University, Ames, Iowa

Japan Society of Civil Engineers, Transactions—Japan Society of Civil Engineers, 1-Chome, Yotsuya, Shinyuko-Ku, Tokyo, Japan

Journal of the Boston Society of Civil Engineers — Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Journal of the American Ceramic Society and Ceramic Abstracts—American Ceramic Society, 4055 N. High St., Columbus 14, Ohio

Journal of the Institution of Engineers Australia—Institution of Engineers, Gloucester and Essex Sts., Sydney, N.S.W., Australia

Journal of the Japan Society of Civil Engineers — Japan Society of Civil Engineers, 1-Chome, Yotsuya, Shinyuko-Ku, Tokyo, Japan

Journal of the Prestressed Concrete Institute — 207 W. Wacker Drive, Chicago, Ill.

Journal of Research—National Bureau of Standards, Washington 25, D.C.

Magazine of Concrete Research—Cement and Concrete Association, 52 Grosvenor Gardens, London, SW 1, England

Materiales Maquinaria y Metodos para la Construccion — Asociacion Tecnica de Derivados del Cemento, Balmes, 163, Barcelona, Spain

Memoires de la Société des Ingénieurs Civils de France — 19 Rue Blanche, Paris 9e, France

Memoirs of the Faculty of Engineering Kyoto University—Kyoto University, Kyoto, Japan

Modern Concrete — 431 S. Dearborn St., Chicago 5, Ill.

Nase Gradevinarstvo — Association of Societies of Civil Engineers and Technicians of Yugoslavia, Kneza Milosa 7, Belgrade, Yugloslavia

National Building Research Institute, Bulletin — South African Council for Scientific and Industrial Research, Pretoria, South Africa

New Technique and Advanced Experience in Construction — Sadovaya 18, Leiningrad, Russia

New Zealand Concrete Construction — New Zealand Portland Cement Association, 102 Featherston St., Wellington, C 1, New Zealand

Nordisk Betong — Nordisk Betonforbund (Nordic Concrete Association), Stockholm 70, Sweden

Novaia Tekhnika i Peredovoi Opyt v Stroitelstve — Moscow, Russia

Oesterreichische Ingenieur-Zeitschrift — Mölkerbastei 5, Vienna I, Austria

Ontario Hydro Research News — Research Division, Hydro-Electric Power Commission of Ontario, 620 University Ave., Toronto 2, Ont., Canada

Pit and Quarry — 431 S. Dearborn St., Chicago 5, Ill.

Polish Technical Abstracts — Centralny Instytut Dokumentacji Naukowo-Technicznej, Aleja Niepodleglosci 188, Warsaw, 12, Poland

Polytechnisch Tijdschrift — 28 Balistraat, The Hague, Netherlands

Precontrainte-Prestressing — 21 Rue Newton, Brussels 4, Belgium

Progressive Architecture — 430 Park Ave., New York 22, N.Y.

Public Roads—Bureau of Public Roads, U.S. Department of Commerce, Washington 25, D.C.

Rakennusinsinööri — Liisankatu 18. D, 49, Helsinki, Finland

Reinforced Concrete Review, The — Reinforced Concrete Association, 94-98 Petty France, London, SW 1, England

Revista Constructiilor si a Materialelor de Constructii — Revistele Tehnice ASIT, Str. Ioan Ghica nr. 3, Bucharest, Rumania

Revista del Colegio de Ingenieros de Venezuela — Praque Los Caobos, Apartado 2.006, Caracas, Venezuela

Revue des Materiaux—Centre d'Etudes et de Recherches de l'Industire des Liants Hydrauliques, 19 Rue Lafayette, Paris 9e, France

Roads and Streets — 24 W. Maple St., Chicago 10, Ill.

Road Research Notes — Road Research Laboratory, Harmondsworth, Middlesex, England

Rock Products — 79 W. Monroe St., Chicago 3, Ill.

Schweizer Archiv für angewandte Wissenschaft und Teknik — c/o Buchdruckerei Vogt-Schild AG, Solothurn, Switzerland

Statens Provningsanstalt, Meddellanden (Bulletins) — Statens Provningsanstalt, Stockholm, Sweden

Stavebnicky Casopsis Slovenskej Akademie Vied — Klemensova 27, Czechoslovakia

Structural Engineer, The — Institution of Structural Engineers, 11 Upper Belgrave St., London, SW 1, England

Swedish Cement and Concrete Research Institute Handlingar (Proceedings)— Svenska Forskningsinstitutet for Cement och Betong vid Kungl. Tekniska Hogskolan I, Stockholm, Sweden

Technical Translations—Office of Technical Services, Department of Commerce, Washington 25, D.C.

Travaux — 6 Avenue Pierre ler-de-Serbie, Paris 16e, France

University of Illinois Engineering Experiment Station, Bulletin — University of Illinois Engineering Experiment Station, 106 Engineering Hall, Urbana, Ill.

Vag-Och Vattenbyggaren — Brunkebergstorg 20, Stockholm C, Sweden

VDI Zeitschrift — Verein Deutscher Ingenieure-Verlag GmbH, Ingenieurhaus, Prinz-Georg-Strasse 77, Düsseldorf, Germany

Waterways Experiment Station, Technical Memorandum—Waterways Experiment Station Corps of Engineers, Vicksburg, Miss.

Zement-Kalk-Gips — Kleine Wilhelmstrasse 7, Wiesbaden 16, Germany

Zement und Beton — Oesterreichischen Betonvereins und Verein der Oesterreichischen Zementfabrikanten, Strohgasse No. 21 A, Vienna 3, Austria

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Bridges

Bridge launcher of light-alloy

The Engineer (London), V. 210, No. 5449, July 1, 1960, p. 33

Reviewed by ARON L. MIRSKY

Brief description of a "launcher" used in the construction of the Wolvercote viaduct on the Southern and Western bypass road at Oxford, without disrupting traffic. Viaduct, with 12 spans varying in length from 40 to 73 ft, is constructed of simply supported precast prestressed concrete beams. Launcher consists of a trussed girder of inverted delta shape, spanning between two movable towers; the second or "follower" tower is anchored to a bridge beam, which acts as a counterweight, and used to move

the trussed beam over the existing road to the leading tower; cradles carrying the bridge beams run along the heavy lower chord ("boom") of the trussed girder.

Discussion (*The Engineer* (London), V. 210, No. 5455 Aug. 5, 1960, p. 240; Aug. 12, 1960, p. 276; Aug. 26, 1960, p. 352) concerns the economics of the launcher, it being pointed out that heavy-duty truck-mounted cranes are available and have been used for such erection chores; one such job, on the Ross Spur motorway over the River Wye, is described and pictured on p. 248 of the Aug. 5 issue of *The Engineer*.

Bridge deck survey

Expanded Shale Clay and Slate Institute, 3rd Edition, Sept. 1960, 24 pp.

In 1953 the Expanded Shale Clay and Slate Institute undertook a survey covering the United States and Canada of bridges whose construction incorporated expanded shale lightweight concrete. The original survey was published in 1954 containing a tabulation of about 40 structures.

This third edition reports on more than 150 bridges. The information is given in tabular form and includes: name of bridge, date of construction, location, owner, engineer, contractor, use and mix, wearing surface, traffic, and condition at time of survey.

Precast prestressed concrete bridges. 1—Pilot tests of continuous girders

PAUL H. KAAR, LADISLAV B. KRIZ, and EIVIND HOGNESTAD, *Journal*, Research and Development Laboratories, Portland Cement Association, V. 2, No. 2, May 1960, pp. 21-37

AUTHORS' SUMMARY

The type of bridge which is under study involves I-shaped girders with a continuous situ-cast deck slab. In current practice, each bridge span is usually constructed of separate precast girders without utilizing the benefits which may result from continuity in the longitudinal direction. This report presents the results of an experimental investigation regarding the feasibility of establishing continuity between precast girders from span to span. Adequate continuity and negative support moment resistance were obtained by placing deformed bar reinforcement in the deck slab across the piers supporting the girders.

Dams

Massive buttress dams

G. S. SARKARIA, *Indian Concrete Journal* (Bombay), V. 34, No. 7, July 1960, pp. 249-256, 275

A detailed and well-illustrated comparative study of buttress versus gravity dams. Shortage of cement and aggregates, remoteness of site, and lower labor wages favor massive buttress dams. Author concludes that today, massive buttress dams, built of plain unreinforced concrete are preferred to the older multiple-arch and Ambursen types of reinforced concrete.

Design of small dams

U.S. Bureau of Reclamation, 1960, 611 pp., $6.50. Available from the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C.

The book is intended to provide engineers with information and data necessary for the proper design of small dams. The data are in a form readily usable by engineers who do not specialize in this field and are intended to simplify design procedures for small earthfill dams particularly.

Only the more common types of small dams now being constructed are discussed. These include concrete gravity, earthfill (rolled-type) and rockfill dams. Emphasis is placed on the design of the common, rolled earthfill type. Small dams are defined as earth- and rock-fill structures with heights less than 50 ft above stream bed, and concrete dams on pervious foundations whose "head" from headwater to tailwater does not exceed 20 ft.

In addition to discussions of the design of dams, spillways, and outlet works, the book contains chapters on project planning, flood studies, foun-

dations and construction materials, diversion during construction, and maintenance and operation. Appendices cover hydraulic computations, construction of embankment, concrete in construction, sample specifications, and other supplementary features.

Bureau of Reclamation personnel prepared the book under the direction of Grant Bloodgood, assistant commissioner and chief engineer, and L. G. Puls, chief designing engineer. More than 30 engineers and many technicians participated in the preparation of the book and in its critical review. Special recognition is given to O. L. Rice, chief of the Dams Branch; H. G. Arthur, supervisor, Design Unit, Earth Dams Section; and E. H. Larson, head of the Manuals and Technical Records section.

An IBM-650 program for the computation of thermal gradients in mass concrete structures

E. E. McCoy and D. L. Neumann, *Technical Report* No. 6-540, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Apr. 1960, 24 pp., $0.50

Authors Summary

A computer program for the computation of the unsteady state temperatures that develop during the construction of a concrete dam in layers (lifts) is described. Information pertaining to the following ambient and structural conditions and thermal properties is necessary for a computation: ambient air temperature and foundation temperature, concrete temperature at time of placement, lift thickness, number of days between lifts, and adiabatic temperature rise and thermal diffusivity of the concrete. The computation may be continued for 50 lifts. The computation is based on a numerical procedure for the solution of the heat-flow equation for one dimension. The space interval is one-quarter thickness of a lift, and the time interval is ¼ day. The program is written for the basic IBM-650 computer with a 2000-word drum. Examples are given to illustrate the types of problems for which the program is suitable.

Design

On the stability of the equilibrium of circular arches subjected to a uniform pressure (Sur la stabilité de l'équilibre des voutes circulaires soumises a une pression uniforme)

E. Baticle, *Le Génie Civil* (Paris), V. 137, No. 10, May 15, 1960, pp. 231-233

Reviewed by Aron L. Mirsky

Author derives expressions for the stability against buckling of fixed-end arches* and for the critical loads. It is pointed out that under certain conditions a pseudo-buckling occurs, which may be injurious.

Design of cylindrical shells considered as folded plates (in Dutch)

H. Grabowsky, *Cement* (Amsterdam), V. 12, No. 7, Feb. 1960, pp. 617-623

Reviewed by John W. T. Van Erp

Second paper on this subject demonstrates the calculation of transverse moments. These and the normal stresses are arrived at by reiterative methods in tabular form. These calculations for a folded plate show a good approximation for a cylindrical shell, provided certain conditions are fulfilled; great length-width ratio, etc.

Prestressed hanging structures (Cubiertas colgantes pretensadas)

David Jawerth and Hans Hottinger, *Bulletin* No. 109, Instituto Tecnico de la Construccion y del Cemento, Madrid, 1960, 38 pp.

Various systems for construction of hanging roofs are defined and described. Special attention is given to the system which consists essentially of two primary cables, within a common vertical plane, and which are curved, each in the opposite sense to the other; the two cables being linked together by a series of diagonal tie cables. The details and statical basis of the system are discussed. The Bulletin also contains the calculation of forces in the various parts of the structure and information on experimental laboratory work which has confirmed theoretical results.

Properties of Concrete

Determination of the compressive stress-strain properties of concrete in flexure

R. G. SMITH, *Magazine of Concrete Research* (London), V. 12, No. 36, Nov. 1960, pp. 165-170
AUTHOR'S SUMMARY

A description is given of an apparatus for the determination of the compressive stress-strain properties of concrete in flexure, based on the principle developed by the Portland Cement Association. The apparatus can be used in a standard universal testing machine with little or no modification. The relatively small size of the test specimen is convenient and economical, especially when materials for models are being tested.

A theoretical analysis is developed which enables the distribution of the stress in the concrete to be determined across the section without reference to the magnitude of the strain. If a value is assumed for the strain at any one stage of loading, the theoretical strains at all other stages may be determined.

Comparison is made between the theoretical strains and the experimental strains obtained during a test on a concrete prism.

General

Lightweight fire protection and the structural engineer

A. R. MACKAY, *The Structural Engineer* (London), V. 38, No. 1, Jan. 1960, pp. 20-25
AUTHOR'S SUMMARY

States that the use of dense concrete casings for the fire protection of structural steelwork is outdated and uneconomical.

Consideration is given to the alternative of using vermiculite-gypsum plaster as the basis of lightweight fire protection; this includes hitherto unpublished data from estimates of the cost of a typical ten-story building showing the financial advantages in favor of lightweight fire protection.

The prospects for a large scale use of vermiculite gypsum plasters as a means of making steel framed buildings lighter and cheaper are considered, and various general recommendations are introduced for the guidance of structural engineers and others who may be involved in work of this nature.

A lateral-force formula based on seismological concepts

FRANK NEUMANN, *Trend in Engineering at the University of Washington*, V. 12, No. 3, July 1960, pp. 4-12
Reviewed by ARON L. MIRSKY

Discusses proposed formula for estimating acceleration and displacement impressed on a structure possessing any period and any damping by a quake of any intensity. Formula is intended to replace those based on the 0.1 g lateral-force concept found in many current building codes.

Some contributions from nuclear power to engineering practice

IAN DAVIDSON, *Proceedings*, Institution of Civil Engineers (London), V. 17 (Session 1959-60), Oct. 1960, pp. 121-136
Reviewed by ARON L. MIRSKY

Discussion of the design and construction of large steel pressure vessels (pp. 122-127) and concrete shields (thermal, biological) (pp. 127-130) leads to a discussion of prestressed concrete pressure vessels, which combine most of the advantages of separate steel pressure vessels and concrete shields but raise some new problems. Basic design data for such a vessel are given in an appendix.

Technical conclusions of the fifth session (CEB) at Vienna, April 1959

English translation of *Bulletin D'Information* No. 19, Comité Européen du Béton, Paris, Apr. 1959, 49 pp. (Limited supply of single copies available from ACI headquarters.)

Another in the series of English translations of reports of the Comité Européen du Béton being made available by the ACI-CEB Collaboration Committee.

This translation includes a summary by CEB President Levi of the general problems of the session plus technical conclusions on simple bending, bond and anchorage reinforcement, cracking, deformations, shear, notation, buckling, and shear.

DISCUSSION, PROCEEDINGS V. 57

Discussion of papers published in the October through December, 1960, JOURNALS appears in the concluding pages of this June issue, as the Institute continues its quarterly publication of discussion. Discussion of papers published January through March, 1961, will appear in Part 2, September 1961 ACI JOURNAL. Discussion of April through June, 1961, papers will appear in Part 2, December 1961 JOURNAL along with index and errata for V. 57.

Discussion of a paper by Howard P. Harrenstien:

Hyperbolic Paraboloidal Umbrella Shells under Vertical Loads*

By ROGER DIAZ DE COSSIO, IGNACIO MARTIN, and AUTHOR

By ROGER DIAZ DE COSSIO†

Professor Harrenstien is to be commended for his paper giving much needed information on the behavior of reinforced concrete hyperbolic paraboloidal shells. It is the contention of this writer, however, that elastic direct stresses are not necessarily important. They are usually small, and in most practical instances are significant only at free edges in compression. One objection to Professor Harrenstien's presentation is that no mention was made of the fact that, since the influence surfaces are drawn on the basis of elastic direct stresses only, and since concentrated loads produce large amounts of bending, equilibrium cannot be satisfied by stresses computed from such surfaces.

Bending stresses and plastic deformations are important, and they should be considered in some degree to obtain a satisfactory design. Much more basic experimental information on the behavior of reinforced concrete shells is still needed before they are designed in such a manner that not only strength requirements are satisfied, but also cracking and deformation lie within tolerable limits.

In the ordinary analysis of a shell, it is assumed that the material behaves in an essentially elastic manner, and that the shell remains true to form during its entire life as a structure. In an actual concrete shell, these two assumptions are not fulfilled completely. From observations of actual shells in the field, the writer has come to believe that by far the largest source of trouble is due to long-time plastic deformation; in other words, from shrinkage and creep under long-time loads. Many shells have been observed that have cracked and deflected badly, especially those with free edges, whether stiffened by edge members or not.

These and similar considerations, plus the interest in this type of structure because of its wide use in Mexico, led the Institute of

*ACI JOURNAL, V. 32, No. 4, Oct. 1960 (*Proceedings* V. 57), p. 385. Disc. 57-18 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 12, June 1961 (*Proceedings* V. 57).

†Member American Concrete Institute, Research Professor, Institute of Engineering, University of Mexico, Mexico D.F., Mexico.

Engineering, University of Mexico, to undertake a program to study shells both analytically and experimentally. As part of the experimental studies, two reasonably large hyperbolic paraboloidal umbrella shells were built in the laboratory, one reinforced throughout, and the other prestressed at the edge members. They have been subjected to their own weight, and only vertical deflections have been measured since they were built. It is intended to load them to failure in the near future. A brief description of the tests and some preliminary results will be given below.*

The shells are 26 ft square in plan, as shown in Fig. A, with a 3-ft rise and 1.2 in. thick, with a 4 x 4 in. stiffening member at the edge, and stiffening members perpendicular to the midpoints of the sides. The shells were designed by ordinary simplified membrane theory, under a total load of 41 lb per sq ft. According to the theory there was a tensile force of about 6.2 tons at the midpoint of the edges. For the reinforced umbrella, this tension was taken by 5 #5 deformed mild steel bars that were cut off towards the ends of the edge. In the prestressed umbrella, an axial compression force of the same magnitude was applied at the edge using post-tensioned unbonded tendons. The

*Felix Candela served as adviser during the design and construction of the umbella shells. Prescon de Mexico provided the prestressing tendons, and also some financial assistance.

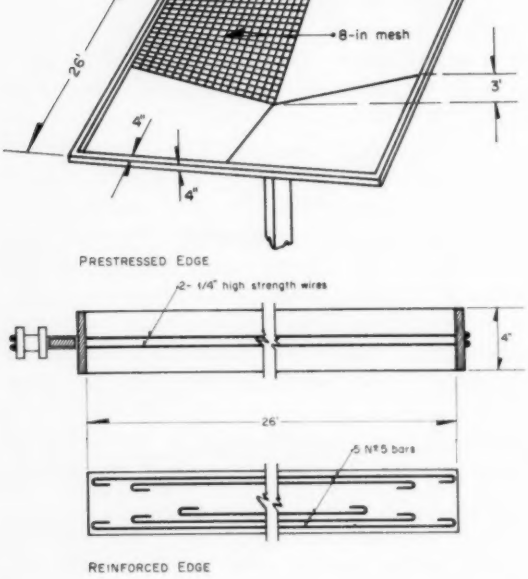

Fig. A—Details of umbrella shells

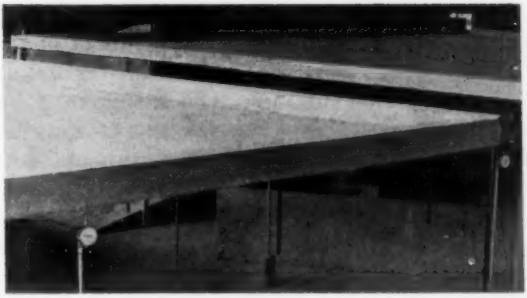

Fig. B — Measurement of vertical deflections

reinforcement throughout both shells consisted of an 8-in. mesh of ¼ in. plain rounds, parallel to the edges.

Fig. B shows a photograph of both umbrellas, and the dial gages from which deflections were recorded daily. Cracking was marked periodically with ink.

As time elapsed, cracks developed parallel to the lines of the reinforcement, and not necessarily under a reinforcing bar. In the reinforced umbrella, cracks extended to a width of about 0.02 in. in 60 days, near the edges and perpendicular to them. Cracking occurred much less in the umbrella with prestressed edges, and was localized in regions closer to the center column. The width of cracks for the same age was considerably smaller. A photograph showing the cracking at the lower surfaces of the reinforced umbrella is presented in Fig. C.

Vertical downward deflections in the reinforced umbrella were substantial: at the end of 60 days the corners deflected an average of about 2 in. The deflections were proportionately smaller towards the center of the edges, and also towards the center column, where they were

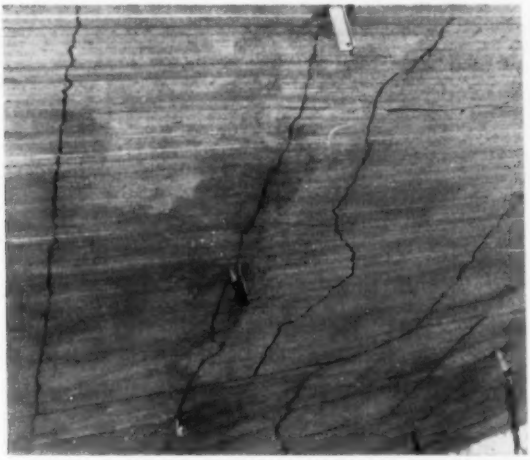

Fig. C — Cracking of reinforced umbrella

almost nil. However, in the umbrella with prestressed edges the corners deflected *upward* an average of about 1.0 in.

It follows that there will be an optimum amount of prestressing at the edges that will be just sufficient to counterbalance plastic and elastic deflections. This amount is apparently smaller than that required to balance the tensile stresses at the edges due to live and dead load.

Deflection-time curves for the corners are shown in Fig. D. At the end of 60 days the downward deflections of the reinforced umbrella were of the order of four to six times the instantaneous deflection when the forms were removed; for the prestressed umbrella the ratio was from two to seven times, but the deflections involved were smaller.

Based on the above discussion, the following conclusions can be drawn:

1. From many observations of shells built in Mexico, it has been found that by far the largest sources of trouble are not the elastic direct stresses but long-time actions, mostly shrinkage and creep under dead load. They will make the shells deflect and crack badly. One word

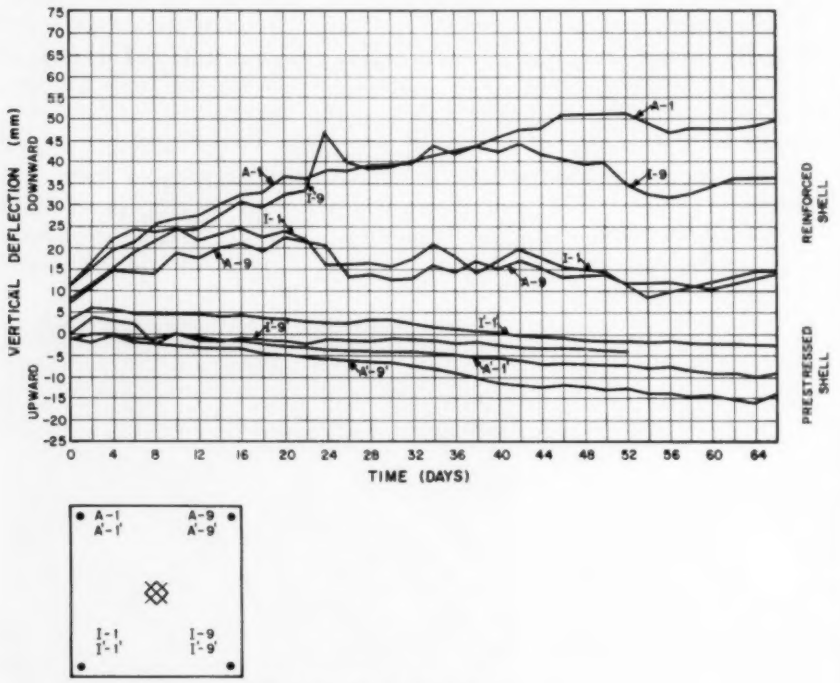

Fig. D—Deflection-time curves
Reinforced shell points: A-1, A-9, I-1, I-9
Prestressed shell points: A'-1', A'-9', I'1, I'-9'

of caution is necessary, however: for shells with free edges in compression, elastic buckling will be critical, and it is then important that the shell remain true to form. Sometimes the neglect of bending stresses may also be critical.

2. From the comparative behavior under dead load of the two umbrella shells built in the laboratory, one reinforced throughout and the other identical to the first one except that the edges were prestressed, it can be concluded that, in general, the presence of a prestressing force considerably improves the deflection and cracking behavior of this type of shell. It can also be said that the optimum amount of prestressing force applied axially to the edges is smaller than that required to balance the tensile stresses due to dead and live load. Since dead load is always present, the prestressing force required will be the one necessary to balance all the dead load tensile stresses at the edges, plus only a portion of the live load.

By IGNACIO MARTIN*

Hyperbolic paraboloidal umbrella shells are economical roofs, because they act as cantilevers from each column, thus giving the minimum bending moment for the distance between columns. For this reason they are used extensively as roofs for industrial buildings. However, in industrial building roofs linear loads are often present, and hence the information contained in this paper should be of great value in the practical design of umbrella type industrial roofs.

The bending stresses cause deflection and deflection in structures subject to axial loads may cause an increase in the bending moments. The plastic deflection due to bending stresses may have been the cause of some recent failures of thin shells, that occurred after 3 months' use. It is regrettable that the author considered that "it is beyond the scope of this investigation to present the resulting bending stresses throughout the shell for the various load points," as that information could be most valuable.

The author is to be congratulated because the investigation presented in this paper is an excellent example of how helpful information can be gathered for the designer at low cost in university laboratories, using sound research ingenuity in the testing procedures.

AUTHOR'S CLOSURE

I wish to express my gratitude to Professor Díaz de Cossío and Mr. Martín for their interest shown by their discussions to my article on umbrella shells.

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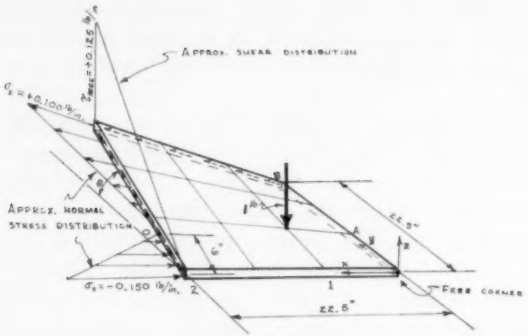

Fig. E—Free body from Fig. 7 (stress contours for 1-lb load at A-1 on 22.5 x 22.5 in. section)

I agree with both individuals that elastic direct stresses are not as important as are the bending and other so-called secondary stresses which are produced by loading, shrink, creep, etc. Mr. Martín has included an interesting observation that stresses produced by bending may lead to undesirable creep and elastic deformations as well inelastic buckling. It has been the author's contention for some time that these bending moments which exist in spite of the accepted membrane theory are and will be the cause of much of our troubles with the behavior of thin shells for the reasons Mr. Martín mentioned.

It appears that a much better picture of the magnitudes of these bending moments is needed so that both short and long-time deflections may be predicted. The author regrets that he was not able to present diagrams of the bending stresses which existed in the umbrella shells that were tested for the original paper. The data from which these results may be produced are presently being published as a bulletin by the Iowa Engineering Experiment Station, Iowa State University, Ames. As soon as funds are available, it is the author's intent to process this data and present the results as stress contours which may be used in conjunction with those found in the paper.

Professor de Cossío is to be commended for initiating a research project to observe the deflection versus time behavior of umbrella shells with edge beams. We of the University of Arizona are looking forward to the completion of this project to compare the results with research that we are currently performing.

The Department of Civil Engineering, University of Arizona, is involved in a model study program which it is hoped will enable the prediction of secondary stresses in hyperbolic paraboloids due to various boundary conditions. Among these various conditions are edge beams of the type tested by Professor de Cossío. Currently, however, no work is planned in the area of prestressed edge beams, although based on Professor de Cossío's tests, it appears that such edge beams may greatly improve the deflection behavior of umbrella shells.

Professor de Cossío states that equilibrium may not be satisfied by stresses computed from the influence surfaces presented in the paper. It is the author's conviction that as long as vertical shearing stresses are small, equilibrium of forces in the vertical direction *must* be satisfied if the data presented is valid. Bending and twisting moments are merely couples and as such do not yield a contribution to the equilibrium condition of $\Sigma F_z = 0$. Bending and twisting moments do, however, affect the equilibrium checks as far as moments are concerned. As an example of how the validity of the influence surfaces may be investigated, consider Fig. E which represents a corner portion of the shell tested in terms of 1 lb at position A-1. From Fig. 7 it is possible to plot the normal and shearing stresses on the exposed faces of this element. Note that from symmetry, only the contribution of the direct stresses on one face need be considered and this effect doubled to obtain the total effect. For ease of subsequent integration and summation operations, these stress distributions were approximated by the linear variations shown. For the element shown, the total vertical component in pounds due to the shearing stress along one face may be computed as

$$\left[\frac{\tau_{max}}{2}\left(\frac{22.5}{\cos\theta}\right)\right]\sin\theta = \frac{\tau_{max}}{2}(22.5\tan\theta) \quad \text{.....(1)}$$

For the shell under consideration $z = xy/180$ where all units are in inches and where the left-hand coordinate system shown in the figure is used.

$$\text{Now} \quad \tan\theta\Big|_{x=22.5} = \frac{\partial z}{\partial y}\Big|_{x=22.5} = \frac{x}{180}\Big|_{x=22.5} = 0.125$$

Then on substitution into Eq. (1) $(0.125/2)\,(22.5)\,(0.125) = 0.176$ lb ↑.

The total vertical contribution due to shearing forces τ on both faces of the element is then $0.176 + 0.176 = 0.352$ lb ↑. As long as the slopes

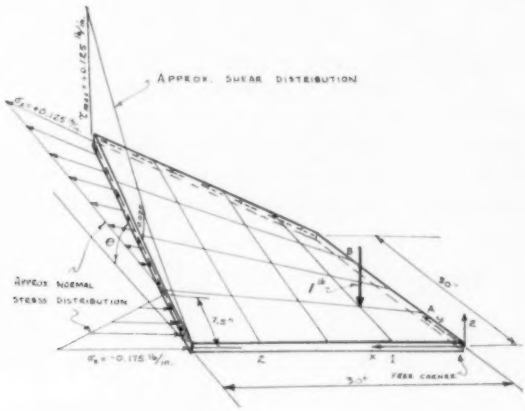

Fig. F—Free body from Fig. 7 (stress contours for 1-lb load at A-1 on 30 x 30 in. section)

remain small, the total vertical component in pounds due to the normal stress σ_x on the face shown may be computed as

$$\int_0^{22.5} \sigma_x \left.\frac{\partial z}{\partial x}\right|_{x=22.5} dy = \int_0^{22.5} \sigma_x \left(\frac{y}{180}\right) dy \quad \text{.....(2)}$$

If σ_x is written as a function of y which represents the linear distribution shown, Eq. (2) yields 0.072 lb ↑ when integrated. Thus the total vertical component of the shear and normal stresses adds up to 0.352 + 0.072 + 0.072 = 0.496 lb ↑. This value is considerably less than 1.00 lb required by equilibrium conditions. Reasons for the discrepancy are:

1. Vertical shearing stresses, which are known to exist in regions of bending, may be present which carry a portion of the 1-lb load. The presence of these stresses affects equilibrium in the z-direction.
2. The experimental data or its results as presented are in error.

Obviously, the author would rather accept Item 1 as being the most logical; however, because of the difficulties which are associated with the measurement of small strains in concrete, Item 2 cannot be disregarded. If Item 1 is significant, the equilibrium equation $\Sigma F_z = 0$ should be more nearly satisfied as free bodies are used which have their edges farther from the load. This is found to be true if we consider the diagram shown in Fig. F which is 30 in. square in plan.

If the same procedure of summing forces in the z-direction as used previously is applied to this free body, it is found that the total vertical component of the shear and normal forces adds up to 0.772 lb which is considerably closer to 1.00 lb than the previously computed 0.496-lb value.

On the basis of these approximate equilibrium checks the author wishes to conclude that the direct stresses presented appear to be reasonable. Quite possibly, when the true bending moments are considered, vertical shears may be predicted by an approach similar to equating $V = dm/dx$ in beam theory. To what extent this requirement improves the vertical equilibrium condition remains to be seen.

In conclusion, I certainly wish to encourage all forms of research which have as their goal the accumulation of information on the behavior of reinforced concrete shell structures. The material in its true nonhomogeneous, nonelastic state is most challenging to everyone from contractors to engineers, architects, and research professors.

Discussion of a paper by Anton Tedesko:

Shell at Denver—Hyperbolic Paraboloidal Structure of Wide Span*

By PETER E. ELLEN and AUTHOR

By PETER E. ELLEN†

The paper by Dr. Tedesko illustrates the necessity to investigate shell roof deflections caused by dead load, concrete drying shrinkage, and creep, which if neglected, can radically alter the stress distribution. The accuracy of shell roof analysis depends principally on the accuracy of the basic assumptions.

A curved shell is able to take load perpendicular to its surface by membrane forces, providing the shell can deform freely under the given set of forces. When no bending or torsion can take place the membrane theory gives a reliable approximation to the actual distribution of internal forces.

In calculating the distribution of forces in an h.p. shell, the membrane theory is first applied and subsequently corrected at the edges for deformation resulting from local stress disturbances. If the shell edge member is rigid and deforms less than the shell proper, bending will occur in the shell adjacent to the edge member. Several methods of determining these bending moments have been advocated but exact analytical solutions are often difficult to obtain and are generally not warranted when other factors such as shrinkage and creep of the concrete are considered.

A simple but sufficiently accurate method is to assume an elemental strip of the shell at right angles to the edge member to be supported by this member and an "elastic foundation,"[1] the stiffness of which is inversely proportional to the radii of shell curvature at that point. The characteristic "wave" of bending moment rapidly decreases in magnitude as it progresses from the edge beam into the shell, according to the stiffness of the shell. These methods have been discussed by Vreedenburgh[2] and can be applied to all forms of shell structure of single or double curvature. This approach presupposes that the edge member is rigid normal to the shell and therefore not deflecting under

*ACI JOURNAL, V. 32, No. 4, Oct. 1960 (*Proceedings* V. 57), p. 403. Disc. 57-19 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 12, June 1961 (*Proceedings* V. 57).

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dead load. A slight deflection would sometimes be beneficial but this is difficult to control or to vary along the length of the member to suit the particular shell characteristics.

The deflections in shell roof edge members can be controlled by introducing prestressing forces in the edge members to counteract the effects of tension and/or shear forces, dead load deflections, resultant bending moments, and torsional strains (e.g., by cables either draped or helical).

In the shell built in Denver, particular attention was paid to the decentering sequence of the precambered formwork to ensure that the shell was not overstressed during construction, yet large deflections of the edge beam members were recorded and some unexpected cracking occurred. The causes of these problems are the self loading of the shell due to its own dead weight and the deflections of the shell edge beam members.

Problems associated with the self loading of the shell can be solved by using a system of jacks at the buttress points to apply forces equal and opposite to the shell dead load forces making the shell carry its own weight without support from the formwork. This eliminates the necessity of formwork camber and allows the removal of formwork without risk of damage to the shell.

Shell edge beam deflections can be closely controlled by using draped prestressing cables in the members, thus eliminating a large percentage of the shell edge disturbances.

In zones where large compressive forces exist, edge beams must be provided to resist these forces, but they add dead weight to the shell edge. If it is assumed that the shell supports this additional weight, uneven distribution of shell forces will result, particularly in large shells. The deflections measured in the Denver shell correspond to those which could be expected from these additional loads. It is these deflections which have caused the shell bending stresses to be greater and extend further into the shell than those which were assumed in the design. Prestressing these members would have eliminated bending due to their own weight and considerably reduced the shell edge disturbances.

The long term effects of concrete drying shrinkage and creep will increase the initial deflections by as much as 250 percent in normal reinforced concrete. However, in shell edge beam members subjected to compression or tension the deflections can be eliminated if the resultant stress distribution across the section is almost uniform, a result which can be achieved by prestressing the members.

The effects of creep due to the axial compressive forces can be reduced by using high strength concrete, (f_c' greater than 5000 psi) and compressive steel reinforcement uniformly distributed across the section. The steel percentage should be as high as possible to reduce

shortening of the member due to creep and to concrete drying shrinkage. Differential strains between the shell and the edge member can be controlled by using suitable distribution reinforcement in the transition zone.

In tension members, the prestressing forces fully counteract the dead load forces and only that percentage of the live load is considered to act continuously (in roofs this is usually assumed to be about 5 lb per sq ft), the remainder of the live load forces is then resisted by non-prestressed reinforcement, either high tensile or mild steel.

The above principles have been applied in a number of shell roof structures built in New Zealand,[3] eliminating dead load deflections and controlling long term movements.

By prestressing, the dimensions of shell edge beam members can be reduced, and the bending moments in the shell adjacent to the edge members can be controlled, often eliminating the necessity of thickening the shell edge.

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1. Timoshenko, S., *Theory of Plates and Shells*, McGraw-Hill Book Co., New York, 1959.

2. Vreedenburgh, C. G. J., "The Hyperbolic Paraboloidal Shell and its Mechanical Properties," *Phillips Technical Revue*, V. 20, No. 1, 1958-1959.

3. Ellen, P. E., "Concrete Shell Roof Construction," *New Zealand Engineering*, V. 15, No. 6, June 1960, pp. 1-7.

AUTHOR'S CLOSURE

Mr. Ellen calls attention to the importance of accurate design assumptions and the advantages of prestressing in reducing deflections and stresses due to deformations. These suggestions are acknowledged together with Reference 3, an article in *New Zealand Engineering*, not readily available to American readers in which Mr. Ellen gives a condensed history and general outline of the use of shell roofs and discusses architectural problems and methods of construction. The prestressing principles of his discussion have been applied to two h.p. shells of umbrella type, built in New Zealand; these shells are quite small as compared to the shell at Denver, one being 18 ft square, the other 20 x 30 ft.

The author believes that there are times when prestressing of a shell may be useful and that it should be investigated. There is a definite justifiable tendency among designers to prestress edge members of shells when these members are subject to tension. The author, however, considers it an oversimplification to think that prestressing is a cure-all which will eliminate all deflections and bending moments. Furthermore, it should be pointed out that the bending moments for an h.p. shell do not decrease in magnitude as rapidly from the edge as

for those shells treated in Mr. Ellen's Reference 1.* Specifically, examining the Denver shell in the light of our 1961 knowledge, a conscientious designer will consider whether or not the marginal ribs of the gable should be prestressed. He will find that prestressing for an edge member which primarily is subject to compressive forces would increase the relative deformation of the edge member with respect to the shell proper. He will most likely come to the conclusion that the size of the edge members cannot be reduced appreciably because of architectural and shell-stability requirements, and that the advantages of prestressing would not be worth the extra cost. Furthermore it would be difficult to justify the additional cost of 5000-psi concrete.

A similar and somewhat larger h.p. shell than the one at Denver is now being constructed for a Colorado auditorium. The designers of that structure decided to prestress the marginal ribs, but in spite of this, these edge ribs turned out to be much heavier than those at Denver. This is perhaps an indication that not a great deal is gained by prestressing these compression members. The elegance of the shell at Denver is due largely to Architect I. M. Pei's careful attention to proportion and handling of detail and to the fullest cooperation between architect and engineer.

Secondary bending moments in the edge member are due not only to the deflection of the edge member but also due to the behavior of the shell. Any special treatment of only the edge member can therefor lead at best to a partial cure. A general statement that creep can substantially be reduced by prestressing does not apply to the gable frames at Denver where any rib shortening produces moments due to the resulting distortion of the frame. Moments due to live load (40 lb per sq ft of snow) cannot be avoided by prestressing for dead load. Practical questions remain such as: To how much trouble should the designer go to avoid only a part of the moments? Is the trouble of jacking at the abutment, as proposed by Mr. Ellen, worth the cost? The post-tensioning of edge members, to have the maximum effect, must be done simultaneously with the decentering; is the cost worth the results? Finally, will the prestressing operation be done without at least temporarily inducing moments of similar magnitude as those to be eliminated?

The author has reason to believe that the insignificant hair cracks in the ribs at Denver, mentioned in his paper, are only detectable at close scrutiny. He believes that the moments due to shell edge restraint are adequately taken care of by the reinforcing provided. He still considers the Denver design features and construction procedures practical and desirable from a United States cost standpoint.

*Mr. Ellen refers to "a simple and sufficiently accurate method" to determine bending moments. The author believes that others will question the accuracy of this method for use with h.p. shells. See Alfred Parme's discussion of an approach by Mario G. Salvadori in "Shells of Double Curvature," *Transactions*, ASCE, V. 123, 1958, p. 1021.

Arbitrary rules should be viewed with caution

An engineer must weigh the facts and make the best decision under the particular circumstances.

Mr. Ellen notes that bending moments should be avoided and recommends prestressing for this purpose. More radical proposals have been made in the past with regard to bending in shells. It should be emphasized that dogmatic rules do not always give the best solution since the engineer must consider all facets of the problem from aesthetics through economics.

As an illustration of how arbitrary rules are sometimes propagated, the author wishes to refer to a series of articles addressed to an architectural audience† which expound a philosophy that could confuse and mislead the readers. In essence it was stated that shells should be used only when what is termed "pure shell action" (membrane state of stress) will result; in other words, bending moments should be eliminated. (As an example, an arch would become "impure" for concentrated or unsymmetrical loads). It is unrealistic to set up a criterion for performance based on so-called pure action. The fact is that "pure action" is the exception rather than the rule in a structure. Structures are composites, and the action of the structure as a whole is not the action assumed by a designer dealing with a simplified mathematical model of the actual structure.

The same series of articles asserts that the use of cylindrical shells is justified only when prestressing is applied. This is like making a statement that the use of reinforced concrete beams is justified only when they are prestressed. Many millions of concrete beams and square feet of cylindrical shells have been built satisfactorily and economically although they are not prestressed and do not fit the specification of pure action.

A better criterion for assessing the success of a structure is: Does the structure satisfactorily and economically fulfill the purpose for which it was designed?

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†Oravas, G. Ar., "Thin Shells: Engineering Fitness and Architectural Form," *Architectural Record*, No. 3 and 4, Mar. and Apr., 1960.

Discussion of a paper by J. L. Waling and Longin B. Greszcuk:

Experiments with Thin-Shell Structural Models*

By MARVIN E. WARNER and AUTHORS

By MARVIN E. WARNER†

Messrs. Waling and Greszcuk have completed an excellent study of the problems involved in using Styrofoam supported by wires for the forming of hyperbolic paraboloid structures. It is well recognized that the forming of a shell represents the major item of cost for such structures. The authors point out the effective insulation and vapor barrier value of the forming material which is left in place, along with the apparent economy realized by eliminating formwork.

A detailed economic study would be desirable to determine if this concept is competitive with the usual scaffolding systems. It would appear that the concept would be economical for long-span hangars, arenas and warehouses, where expensive scaffolding required in the conventional system could be eliminated, or substantially reduced.

The authors make no mention of the material proposed for edge beams, although they used structural steel sections in their models. It would appear that the use of precast-prestressed concrete girders, or built-up steel box girders would be economical for full-scale structures. This would be particularly true for the exterior edges where considerable lateral stiffness is required to resist the tension wire forces.

While the qualities of Styrofoam are well recognized and proved, it probably would be advantageous to consider other materials which combine the qualities of insulation with better sound attenuation such as foamed glass, or corkboard products. It is also possible that a foamed polystyrene which has better acoustical properties and fire resistance has been or will be developed.

It should be emphasized that the use of any of these forming materials will require a thin concrete coating as outlined in the paper, not only for prevention of excessive deflections but also to provide a working surface for the placing of reinforcement, electrical conduits, etc. It has been the writer's experience that when such insulating materials are

*ACI JOURNAL, V. 32, No. 4, Oct. 1960 (*Proceedings* V. 57), p. 413. Disc. 57-20 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 12, June 1961 (*Proceedings* V. 57).

†Roberts and Schaefer Co., New York, N. Y.

used as form liners they have the disadvantage of providing a poor working surface, particularly when wet. The legs of reinforcing bar supports tend to sink into the soft material and concrete vibrators tend to damage the material. With the concept outlined in the paper, such considerations become more important and the thin concrete coating becomes a necessity.

The writer has had the privilege of working with Dr. Waling on a preliminary study covering various configurations for a hangar similar to that shown in Fig. 14, and would certainly encourage further research and tests on the subject.

AUTHORS' CLOSURE

The authors are pleased to see the discussion by Mr. Warner. The previous thin-shell design work of Mr. Warner and his colleagues serves as excellent background for evaluating the new ideas presented in the paper.

The need for reliable cost data on the new shell-forming concept has been recognized from the early period of the research work. However, the research staff has felt that the only way to obtain reliable and publishable cost data on a new and different construction procedure of this type is through an actual construction project. Two such projects are now "on the drawing boards" and are scheduled for construction during the summer. It is the intention of the designers to obtain reliable cost data for these two projects and to make the data available through publication.

Mr. Warner raised the question concerning the material proposed for edge members. Several possibilities, including precast and/or prestressed concrete, built-up steel, or combinations of steel and concrete, seem appropriate. For the two projects now in design, built-up steel sections designed to carry essentially the dead load of the shell are to serve also as the forms for the addition of concrete placed integrally with the concrete shell. The combination of steel and concrete in the edge members would carry the forces induced by dead load plus live load. The open steel forms allow for easy placing and excellent anchorage of the shell reinforcement to the edge members through concrete bond. The side of the steel edge members adjacent to the Styrofoam shell is to be fabricated as a warped surface to be approximately perpendicular to the shell surface all along the edge member. This will facilitate the placing of the Styrofoam wedges. Fig. A shows a typical ridge member between two adjoining hyperbolic paraboloid segments and a typical edge member in their places in the shell structure.

The proposed built-up steel edge members do have considerable lateral stiffness. However, this is somewhat of secondary importance since the wire tensile forces are essentially balanced by the precompression

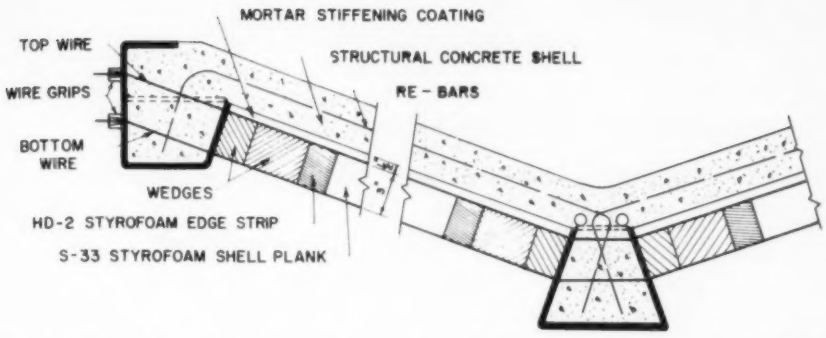

Fig. A—Typical section at an edge member (left) and a ridge member (right)

of the Styrofoam surface. The edge members transmit Styrofoam precompression forces back into wire tensions. Any unbalance between these two sets of forces would be resisted by the transverse strength of the edge members.

It is hoped that other engineers and builders will further develop the new concept described in the original paper. The authors are indeed grateful for the comments of Mr. Warner and for his encouragement toward further research on the subject.

Discussion of a paper by Boris Bresler:

Design Criteria for Reinforced Columns under Axial Load and Biaxial Bending*

By S. CONSTANTINESCU-CATUNESTI, DAVID R. ESTY, PHILLIP GOULD, F. N. PANNELL, and AUTHOR

By S. CONSTANTINESCU-CATUNESTI†

This discussion has two aims: (1) to generalize Eq. (4) in a similar manner as for the discussion on the design under axial load and biaxial bending[13,14] and (2) to suggest a method for further investigations on the subject.

(1) Eq. (4) was established for compression by Nikitin‡ from which Nicula§ developed the following analysis.

Let f_c' be the maximum corner stress of a rectangular section under a load carrying capacity P_{xy}, with the eccentricities e_x and e_y.

Then

$$f_c' = \frac{P_{xy}}{A} + \frac{P_{xy}\,e_x}{W_x} + \frac{P_{xy}\,e_y}{W_y} = \frac{P_{xy}}{A}\left(1 + \frac{e_x}{r_x} + \frac{e_y}{r_y}\right) \quad (8)$$

where

$$r_x = \frac{W_x}{A} \quad \text{and} \quad r_y = \frac{W_y}{A}$$

On the other hand, let P_x and P_y be the load carrying capacities under compression with uniaxial eccentricities e_x and e_y, respectively.

Now

$$f_c' = \frac{P_x}{A} + \frac{P_x\,e_x}{W_x} = \frac{P_x}{A}\left(\frac{e_x}{r_x} + 1\right) \quad (9)$$

$$f_c' = \frac{P_y}{A} + \frac{P_y\,e_y}{W_y} = \frac{P_y}{A}\left(\frac{e_y}{r_y} + 1\right) \quad (10)$$

and let P_o be a load carrying capacity without eccentricities.

$$f_c' = \frac{P_o}{A} \quad (11)$$

*ACI JOURNAL, V. 32, No. 5, Nov. 1960 (*Proceedings* V. 57), p. 481. Disc. 57-23 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 12, June 1961 (*Proceedings* V. 57).

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‡N. V. Nikitin used this equation at the ZNIPS, Moscow.

§I. Nicula established this equation in his lectures to the students of civil engineering, Bucharest.

From Eq. (9), (10), and (11) we obtain the following relationships:

$$P_x = \frac{f_c' A}{\frac{e_x}{r_x} + 1} = \frac{P_o}{\frac{e_x}{r_x} + 1} \quad \text{or} \quad \frac{P_o}{P_x} = \frac{e_x}{r_x} + 1 \qquad (12)$$

$$P_y = \frac{P_o}{\frac{e_y}{r_y} + 1} \quad \text{or} \quad \frac{P_o}{P_y} = \frac{e_y}{r_y} + 1 \qquad (13)$$

Now combining Eq. (12) and (13) with Eq. (8) and (11) we obtain

$$\frac{P_o}{P_x} + \frac{P_o}{P_y} = \left(\frac{e_x}{r_x} + \frac{e_y}{r_y} + 1\right) + 1 = \frac{A f_c'}{P_{xy}} + 1 = \frac{P_o}{P_{xy}} + 1$$

from which

$$\frac{P_o}{P_x} + \frac{P_o}{P_y} - 1 = \frac{P_o}{P_{xy}}$$

and finally

$$\frac{1}{P_x} + \frac{1}{P_y} - \frac{1}{P_o} = \frac{1}{P_{xy}} \qquad (14)$$

(2) The variation of S_2 $(1/P_u,x,y)$ results in a transformation of the failure surface S_1 (P_u,x,y) which is a result of the variation of Eq. (15).

$$f_c' = \frac{P_x}{b\,d}\left(1 + \frac{6\,e_x}{d}\right) \qquad (15)$$

Now if f_c', P_x, b, and d are given, Eq. (15) becomes:

$$P_x = \frac{A}{B + e_x} \quad \text{and similarly} \quad P_y = \frac{A'}{B' + e_y}$$

where A, A', B, and B' are constants.

The curves representing P_x and P_y were plotted in Fig. 1 on the planes xOP and yOP.

Then

$$\frac{1}{P_x} = \frac{B + e_x}{A}$$

which is a straight line, AC, represented in Fig. 4.

Similarly

$$\frac{1}{P_y} = \frac{B' + e_y}{A'}$$

which is the straight line BC. The reciprocal surface S_2 $(1/P_u,x,y)$ is limited by the generating lines AC and BC, and is probably a doubly-curved surface. This could be a possible starting point for a further study of the surface equation.

Eq. (2) may be written in a simpler form for the plane ABC.

$$\frac{x}{x_A}(z_c - z_A) - \frac{y}{y_B}(z_B - z_C) + z - z_C = 0$$

Eq. (15) gives the greater value out of the two possible equations; this is true whether the external force is a compressive or a traction force; the difference lies only in the sign of the P value.
Hence:

$$\sigma_{effective} = \frac{\pm P}{b\,d}\left(1 + \frac{6\,e}{d}\right)$$

It follows that the same reasoning as outlined in (2) may be repeated for an external traction force, and one will arrive at Eq. (4) which is then applicable for both compressive and tensile loads.

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By DAVID R. ESTY*

The author has proposed two criteria for design of columns subjected to compression combined with biaxial bending. The writer had the privilege of working with Professor Bresler on this topic and will present in this discussion further verification of the proposed criteria.

Notation

Notations and units used are those of the paper. Additional notations are: a and b, column dimensions; A_{sb}, area of steel per reinforcing bar; N, number of reinforcing bars per column; and e_u, concrete ultimate strain.

ANALYSIS

For any given column, the load contour graph may be plotted by assuming successive locations of the neutral axis and computing P_u and its corresponding eccentricities. This plot may then be used to determine the values of α and β for Method B and also to compute check examples of Eq. (4) of Method A. Fig. A shows a typical load contour plot.

To provide sufficient data to verify the proposed design methods the IBM 704 digital computer was used to perform the numerous calculations. To indicate the magnitude of the calculations that were required, for all columns together a total of 15,000 values of load with

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their corresponding eccentricities were computed. Approximately 4 hr of production time was used on the computer.

Table A and Fig. B show details of the sections selected for analysis. These sections provide a variety of sizes and steel percentages. Sections 1-11 and 13 have a symmetrical arrangement of reinforcement. Section 12 has unequal amounts of steel in adjacent faces. Section 13 is identical to Section 2 except that all steel is concentrated at the corners. In all cases cover over main reinforcement is 2 in., and the reinforcing bars are equally spaced along any face of the column.

Table B shows the properties of materials selected for analysis. The effect of three variables, viz., ultimate concrete strain, ultimate concrete strength, and steel yield stress, is included in this study. A modified Jensen's idealized stress-strain relationship is assumed. The departure from Jensen's relationship is in the values of ultimate concrete strain, which in this study is assumed independent of concrete strength. The lower value of strain used is computed from Jensen's formula of $f_c' = 3.0$. The higher value of ultimate strain was arbitrarily chosen to be greater than a probable design value. The modulus of elasticity of

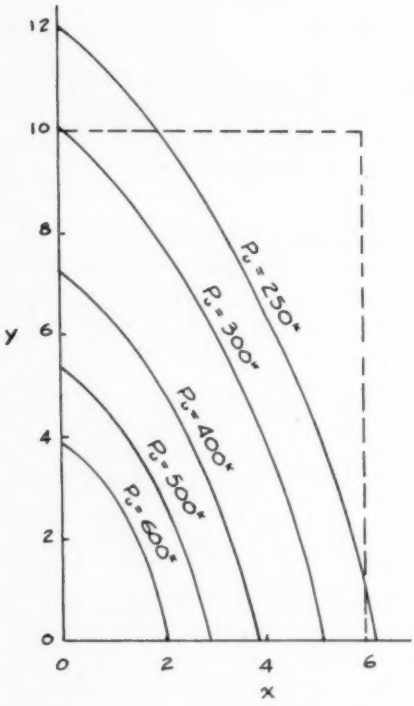

Fig. A—Load contours, Columns 3-1

TABLE A — SECTIONS

Section	a, in.	b, in.	N	A_{sb}, sq in.	P, percent
1	15.00	15.00	4	0.60	1.07
2	15.00	15.00	8	1.27	4.51
3	12.00	20.00	8	0.60	2.00
4	12.00	20.00	8	1.27	4.23
5	24.00	24.00	12	0.78	1.50
6	24.00	24.00	16	1.56	4.00
7	24.00	36.00	16	0.78	1.45
8	24.00	36.00	20	1.56	3.61
9	36.00	36.00	12	1.56	1.45
10	36.00	36.00	32	1.56	3.85
11	36.00	48.00	16	1.56	1.45
12	30.00	42.00	16	1.56	1.98
13	15.00	15.00	4	2.54	4.51

TABLE B — CASES, PROPERTIES OF MATERIALS

Case	f_c', ksi	f_y, ksi	e_u	E_c, ksi $\times 10^{-6}$
1	3.00	50.00	0.00232	3.6
2	3.00	50.00	0.00400	3.6
3	5.00	50.00	0.00232	4.3
4	5.00	50.00	0.00400	4.3
5	3.00	33.00	0.00232	3.6
6	3.00	33.00	0.00400	3.6
7	5.00	33.00	0.00232	4.3
8	5.00	33.00	0.00400	4.3

concrete is assumed dependent on concrete strength and is computed from Jensen's formula. The modulus of elasticity of steel is taken as 30×10^6 psi. The usual assumptions of no slippage, plane sections remaining plane, and negligible concrete tensile strength are made. No reduction of the ultimate concrete strength to $0.85f_c'$ is used. Therefore the strengths used, 3.0 and 5.0 ksi, would in actual practice correspond to higher values of test cylinder strength.

Columns in this paper will be identified by two numbers, the first identifying the Section as shown in Table A, the second identifying the Case, i.e., properties of materials, as shown in Table B. Thus Column 3-1 is identified as the column with size and reinforcing as in Section 3 and analyzed with the properties of materials as shown by Case 1.

RESULTS

Method A

To check the validity of Eq. (4), values of P_u, x, and y are chosen from the computer output data. From plotted P_x versus x and P_y versus y curves values of P_x and P_y at the selected x and y eccentricities are obtained. P_i is then computed from Eq. (4).

There were 274 values of P_i computed and compared with P_u. For each column approximately three random locations of P_u were chosen.

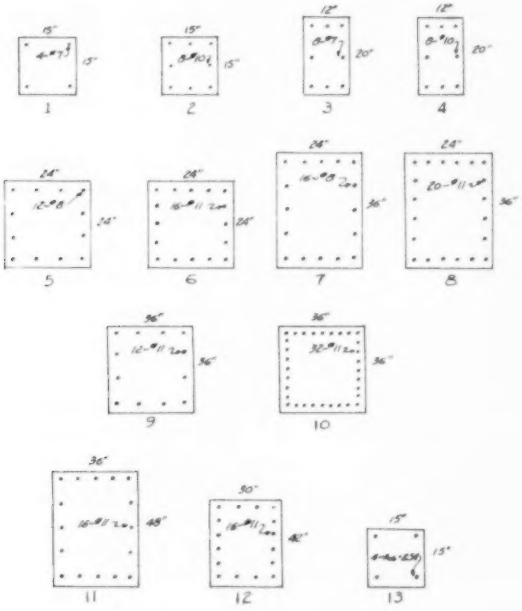

Fig. B—Sections

TABLE C — VALUES OF α

Case	Section										
	1	2	3	4	5	6	7	8	9	10	13
1	1.36	1.16	1.30	1.15	1.34	1.21	1.36	1.20	1.36	1.25	1.14
2	1.41	1.42	1.50	1.44	1.53	1.49	1.57	1.47	1.60	1.51	1.18
3	1.32	1.17	1.30	1.18	1.33	1.22	1.36	1.24	1.35	1.24	1.13
4	1.50	1.43	1.59	1.45	1.52	1.43	1.60	1.50	1.56	1.52	1.24
5	1.48	1.32	1.45	1.36	1.50	1.43	1.50	1.43	1.49	1.46	1.22
6	1.49	1.57	1.65	1.60	1.66	1.58	1.67	1.62	1.66	1.61	1.25
7	1.45	1.36	1.47	1.36	1.43	1.37	1.42	1.39	1.45	1.38	1.27
8	1.50	1.51	1.65	1.62	1.58	1.58	1.66	1.65	1.64	1.56	1.30

In 215 cases P_t was less than P_u, i.e., conservative, and in 59 cases P_t was greater than P_u, i.e., unconservative. The following summarizes results:

Maximum $\dfrac{P_t}{P_u} = 1.142$

Minimum $\dfrac{P_t}{P_u} = 0.765$

Average $\dfrac{P_t}{P_u} = 0.964$

Average deviation $\dfrac{|P_t - P_u|}{P_u} = 5.8$ percent

The largest deviations occur, as might be expected, at small values of P_u. This can, in part, be attributed to the difficulty of exactly defining the load versus eccentricity curves at small values of P. No general trend of deviation is apparent from the results. There is no specific location on a column face where the unconservative deviations predominate.

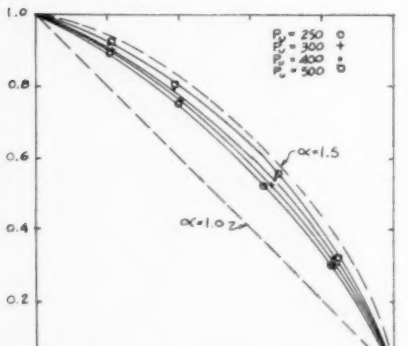

Fig. C—Dimensionless load contours, Column 3-I

Method B

In a symmetrical column the values of the exponents α and β in Eq. (6) are equal. Fig. C shows typical dimensionless load contours for an unsymmetrical column with ideal load contours at $\alpha = \beta = 1.0$ and 1.5 also shown. Comparison of the actual to the ideal load contours shows that the assumption $\alpha = \beta$ is reasonable for an unsymmetrical section, and that the use of Eq. (7) is justified.

It is also seen from Fig. C. that one dimensionless load contour, i.e.,

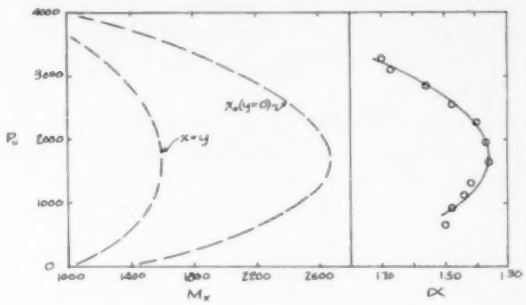

Fig. D—Column 9-I, P_u versus M and α

one value of α, will not exactly define the ultimate capacity of a given column. An analysis of the computed data revealed that the values of α for each column varied considerably, the minimum occurring at values of P_u at or near the balanced design P_u. Fig. D shows P_u versus M curves compared with α versus P_u curve. At large eccentricities the deviation in the value α from the minimum is approximately 20 percent, while at small eccentricities the deviation is as much as 40 percent. The α values shown in Table C and plotted on Fig. E are the minimum values for each section. Use of the minimum value to develop design criteria will lead to a conservative method in all cases.

Inspection of the α values in Table C reveals that a change in concrete strength has little or no effect on the value of α. This will therefore, in this report, be neglected as a variable, and Cases 1 and 3, 2 and 4, 5 and 7, and 6 and 8 are combined on Fig. E. Comparison of α values for Sections 7 and 9 (same steel percentage, p) indicates that the shape of the cross section, a/b ratio, has little or no effect on the value of α, and this will also be neglected as a variable in this report. Section 12 with unsymmetrical reinforcement was not thoroughly analyzed for this report but a check of two cases compared favorably with other data. The ratio of reinforcement in adjacent faces of Section 12 is not from unity, and it might be expected that if this ratio varied considerable from unity its effect could be important.

The columns with the steel concentrated at the corners, Sections 1 and 13, show a marked deviation from the normal trend as shown on Fig. E. The low values of the exponent α obtained for cases with the steel concentrated at the corners can be easily explained. Consider the case of a column with eight bars versus one with four. In uniaxial bending with the neutral axis at the axis of symmetry, one-eighth of the steel is not effective in the eight bar column, but in the four bar column all steel is in the yield range. In biaxial bending when the neutral axis is close to the center approximately one-half of the steel is not effective or at least is not in the yield zone. The result, and the reason for the low α values, is that concentration of steel at the corners is not

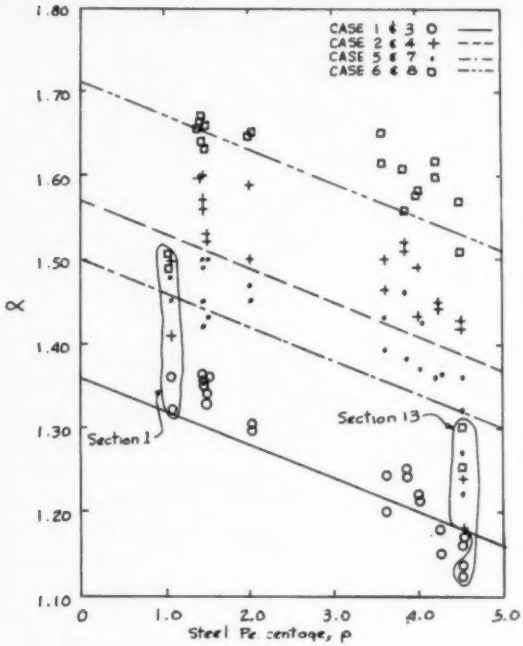

Figure E—α versus *p*

as effective in increasing the ultimate strength in biaxial bending as it is in uniaxial bending.

Neglecting for a moment the α values obtained from the columns with steel concentrated at the corners (Sections 1 and 13, circled on Fig. E), a general trend is apparent. A linear relationship which roughly approximates the lower bounds of the α values of Sections 2-10 is as follows:

$$\alpha = 1.48 + \frac{.824}{100}(15.2\, e_u - f_y - 4.85\, p) \qquad (16)$$

where e_u is in in. per in. $\times 10^3$, f_y is in kips per sq in., and p is in percent. This is plotted on Fig. E for the four combinations of variables studied.

The number of columns analyzed with only corner steel is not sufficient to formulate definite recommendations at this time.

SUMMARY

Two methods of design of reinforced concrete columns under biaxial bending and axial load have been investigated. Neither relieves the designer from selecting trial sections, although once a trial section is selected, the work necessary to determine the section adequacy is greatly reduced. Discussion of the results by individual method follows.

Method A

The results obtained indicate that the values of P_i computed from Eq. (4) compare favorably with the calculated theoretical values, the average $P_i/P_u = 0.964$.

Some of the deviation found is inevitably due to errors in plotting and interpolating the P versus X_o and/or Y_o curves. The majority of the deviations are on the conservative side; however, there are enough unconservative deviations (21.5 percent) to cause concern.

Method B

For columns with distributed steel (eight or more bars) Eq. (16) can be used to determine the value of α in Eq. (7). Now Eq. (7) can then be used to determine the adequacy of a given trial section. In most cases, the use of Eq. (7) and (16) will yield conservative results. As the deviation of the ideal load contour from the actual load contour varies with column and material properties and also with the position of the load contour on the column, no quantitative analysis of the deviation is presented.

Although Eq. (16) is presented as producing reasonably accurate load contours it is recommended that further work be done to more precisely determine the effect of the variables studied in this report, including ultimate concrete strength and steel distribution. Other variables not included in this study are the steel cover d/t and assumed stress-strain characteristics.

Acknowledgments

The writer wishes to express his appreciation to Prof. B. Bresler, University of California, who aroused the writer's interest in the subject and under whose supervision the study was conducted, to Prof. C. F. Scheffey, University of California, who greatly aided the writer in the programing of the computations for the IBM 704 digital computer, and to the personnel of the University of California's computer center, who generously allowed the use of the computer.

By PHILLIP GOULD*

The writer would like to examine loading conditions affecting the validity of Eq. (4), $\frac{1}{P_i} = \frac{1}{P_x} + \frac{1}{P_y} - \frac{1}{P_o}$, for predicting the ultimate strength of a column subjected to axial load and biaxial bending. The observations are concerned with computing the correct values of P_x and P_y to be used in the equation and are, therefore, equally applicable when computing the strength of columns subjected only to uniaxial bending.

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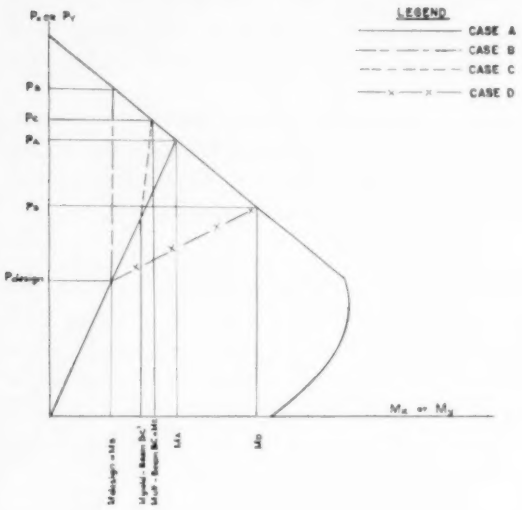

Fig. F — Hypothetical column interaction diagram

Although the equation $\frac{1}{P_i} = \frac{1}{P_x} + \frac{1}{P_y} - \frac{1}{P_o}$ appears to be a good approximation for evaluating the load carrying capacity of a reinforced concrete column subjected to axial compression and biaxial bending, attention must be given to the eccentricities x and y that are used in computing P_x and P_y, respectively, for a given column. Since the axial load at which a column fails is dependent on the moment acting at the time of failure, variations in x or y can have an effect on P_i.

A usual assumption for x or y would be to consider that the eccentricities are unchanged from the design values so that P_x or P_y would be computed using x or y equal to M_{design}/P_{design}. This corresponds to Case A on the hypothetical interaction diagram shown as Fig. F. The possibility exists, however, that the loading to ultimate may follow a different pattern (eccentricity) than the loading to design conditions. To determine an actual factor of safety against failure which might arbitrarily be defined as P_i/P_{design}, the actual loading of the structure to failure must be considered.

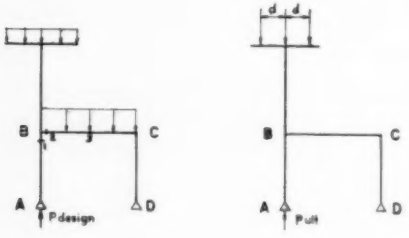

Fig. G — Design loading (left) and added ultimate loading (right)

Fig. H — Design loading (left) and added ultimate loading (right)

If the axial load were to increase without proportionate increase in bending moment, the eccentricity would decrease and the resulting P_x or P_y would be greater than the value calculated by the constant eccentricity to failure assumption (Case A) and result in an increased P_i. This might occur when the structure was subjected to a loading such as that illustrated in Fig. G which would increase the axial load on Column AC without increasing the moment. The behavior of this structure is illustrated by Case B of Fig. F.

Another possibility of increased axial load without proportionate increase in bending moment is illustrated in Fig. H. If the proportions of the frame were such that Beam BC would first yield at Sections 2 or 4 and still have additional moment carrying capacity at Section 3, the beam could carry additional load until the ultimate moment was reached at Section 3. Once the hinge forms at Point 2, the moments at 1 and 2 will remain almost constant although the axial load will increase as the beam is loaded to failure. If the column were to fail before the beam became a mechanism, but after the hinge had formed at Section 2, the eccentricity would be reduced after the moment on the column equaled the yield moment on the beam. This is true only if the beam section at 2 has sufficient rotational capacity to allow the redistribution to take place without crushing. The behavior of Column AB is shown as Case C on the interaction diagram, Fig. F.

On the other hand, if the axial load were held constant or increased slowly while the moment on the column was substantially increased, the value of x or y would be increased as shown by Case D on the interaction diagram, Fig. F. This would result in an overestimation of P_x and/or P_y and P_i using the constant eccentricity assumption and a corresponding overestimation of the safety factor. This type of behavior might occur when unexpected lateral loadings occured on the columns such as in Fig. I.

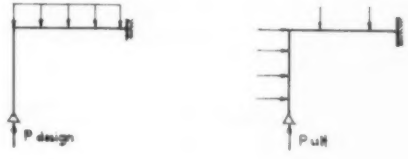

Fig. I — Design loading (left) and added ultimate loading (right)

Although some of the possibilities discussed here might seem rather remote, they illustrate that a change in the manner of loading can alter the behavior of a reinforced concrete column subjected to uniaxial or biaxial bending.

By F. N. PANNELL*

Professor Bresler's interpretation of two equations as approximations to a failure surface is interesting. His conclusions, however, cannot be allowed to pass without comment.

Consider the equation

$$\frac{1}{P_u} = \frac{1}{P_x} + \frac{1}{P_y} - \frac{1}{P_o} \qquad (17)\dagger$$

As Professor Bresler has shown, it is at best a plane approximation to a curved surface, and only in rare cases will it give an accurate prediction; moreover it has no means of correcting for the known deviations of contour of the failure surface. In general, its inaccuracy will increase as the angle of the failure load to one of the major axes increases, becoming a maximum when the column load lies roughly on the column diagonal; it is also unreliable at high eccentricities of load.

To investigate the accuracy of Eq. (17) 24 cases from the writer's original case-analysis were recalculated with it. Almost any interaction equation will give substantially accurate results as the eccentricities x, or y, or both, tend to zero; the cases have therefore been restricted to those in which P_u/P_o is less than 0.8, and in which the transformed angle θ between the failure load plane and the y-axis is between 20 and 70 deg.

The results P_i/P_u predicted by Eq. (4) are given in Column 10 of Table D. The prediction P_g/P_u given by

$$M_y = \frac{M_y \sec \theta}{1 - N \sin^2 2\theta} \qquad (18)$$

is included for further comparison, in Column 9.

It is clear that Eq. (17) is by no means accurate. Its simplicity in use is also open to question. The design problem is to choose a section carrying a known load acting at a known radial eccentricity. For an equation such as Eq. (17) this can only be done by a process of trial and error, with no sound guidance as to the appropriate size to take for each succeeding trial. When using Eq. (18), however, provided the original choice of section is not too far out, the appropriate steel ratio for the correct solution may be read directly off the design curve, without reiteration.

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†Note that Eq. (17) is identical to the author's Eq. (4) except that $1/P_u$ serves as an approximation to $1/P_i$ as shown by the author.

The "directness" of the equation is also a questionable merit. No practical designer would choose an inaccurate basic approximation to the solution of a complex problem in preference to a result read off an accurately computed set of graphs. Moreover, if it is intended to imply that Eq. (17) can be used without design charts, direct from the basic equations, then it is no longer simple; if it is also proposed to tackle columns with steel distributed along two opposing faces or along all four faces, then such an approach would be extremely tedious.

Professor Bresler claims that Eq. (17) is applicable to columns with unequal steel in adjacent faces, but produces no evidence to support this. There appears to be no reason why the equation should be any more successful here than it is for the case of equal steel in each face; and its standard of accuracy for that appears, from Table D, to be unacceptable.

The equation

$$\left(\frac{M_x}{M_{xo}}\right)^{\alpha} + \left(\frac{M_y}{M_{yo}}\right)^{\beta} = 1.0 \qquad (5)$$

is an allied form of Eq. (18). In assuming the Index α to be an empirical constant depending on column characteristics only, a fundamental error is made. The Index α is primarily a function of $P/f_c'bd$ and sec-

TABLE D — RESULTS FROM INTERACTION EQUATIONS

	$\frac{rf_y}{f_c''}$*	$\frac{P}{f_c''bd}$	x/b	y/d	$\frac{P_x}{f_c''bd}$	$\frac{P_y}{f_c''bd}$	$\frac{P_o}{f_c''bd}$	$\frac{P_y}{P_x}$	$\frac{P_i}{P_x}$
1	0.8	0.709	0.1745	0.468	1.225	0.777	1.76	0.99	0.92
2	1.6	0.893	0.207	0.629	1.660	0.952	2.52	0.98	0.89
3	0.2	0.0335	1.295	2.845	0.098	0.0344	1.19	1.01	0.78
4	0.2	0.0482	1.60	1.82	0.072	0.0605	1.19	1.00	0.70
5	0.2	0.1875	0.336	0.725	0.582	0.230	1.19	0.98	1.02
6	0.2	0.3582	0.2765	0.3595	0.652	0.560	1.19	0.98	1.12
7	0.8	0.0486	2.06	6.6	0.190	0.052	1.76	0.97	0.86
8	0.8	0.3489	0.4625	0.804	0.781	0.540	1.76	1.00	1.12
9	1.6	0.3365	0.726	1.43	0.863	0.527	2.52	1.02	1.12
10	0.2	0.804	0.1405	0.0915	0.877	0.975	1.19	1.00	0.94
11	0.2	0.788	0.0804	0.166	1.00	0.828	1.19	1.00	0.93
12	0.2	0.530	0.198	0.249	0.770	0.689	1.19	0.99	0.99
13	0.8	1.114	0.1635	0.1285	1.250	1.340	1.76	1.01	0.92
14	0.8	1.051	0.0985	0.241	1.422	1.087	1.76	1.00	0.90
15	0.8	0.619	0.362	0.448	0.898	0.799	1.76	1.00	0.90
16	1.6	1.528	0.1792	0.155	1.745	1.825	2.52	1.01	0.90
17	1.6	1.402	0.1118	0.2975	1.977	1.435	2.52	1.00	0.89
18	1.6	0.738	0.351	0.639	1.329	0.940	2.52	1.00	0.95
19	0.2	0.0654	0.910	1.496	0.142	0.065	1.19	1.02	0.71
20	0.2	0.0482	1.70	1.70	0.055	0.055	1.19	1.01	0.59
21	0.2	0.1694	0.422	0.718	0.378	0.196	1.19	0.99	0.85
22	0.2	0.3588	0.314	0.314	0.528	0.528	1.19	1.00	0.95
23	0.8	0.3513	0.612	0.612	0.608	0.608	1.76	0.99	1.05
24	1.6	0.3413	1.030	1.030	0.613	0.613	2.52	1.00	1.02

* $f_c'' = 0.85\, f_c'$.

TABLE E — COMPARISON WITH AUTHOR'S TEST RESULTS

	F	$\phi\, b/d$	N	N'	P_s/P_u	
					Eq. (18)	Eq. (20)
1	1.318	—	—	—	1.04	
2	1.315	—	—	—	1.05	
3	1.126	—	—	—	1.02	
4	1.116	—	—	—	0.98	
5	1.137	1.094	0.165	0.85	1.06	1.06
6	1.127	1.086	0.05	0.25	1.01	1.01
7	1.129	1.088	0.08	0.4	1.06	1.02
8	1.127	1.086	0.10	0.5	0.95	0.92

ondarily of rf_y/f_c'. The ratios of α quoted by the author in Table 1 are therefore meaningless. The derivation of α and its values for any rectangular symmetrical four bar column which can be taken as having a value of $e_u E_s/f_y$ in the region of 1.8 are given elsewhere.[15]

Written in the form of Eq. 5 the equation suffers from the same disadvantage in finding a section as Eq. (17). To enable it to be used with the same facility as Eq. (18) it should be rewritten in a form such as

$$M_s = [(M_r)^e + (\phi M_s)^e]^{1/e}$$

or

$$M_s = M_r\,(1 + \tan^e\theta)^{1/e} \qquad (19)$$

where $\phi = M_{fy}/M_{fx}$ and $\tan\theta = \phi x/y$.

Equations in angular form are advantageous for defining failure surfaces; they give a physical interpretation to the surface which enables its properties to be more readily understood, and they are useful when dealing with sections having unequal steel in adjacent faces.

Yet another equation, which has an advantage over Eq. (19) in that it does not contain non-integral indices, is

$$M_s = M_r \sqrt{\sec^2\theta + N' \tan\theta} \qquad (20)$$

Neither Eq. (19) nor (20), as has been shown,[15] is as accurate as Eq. (18). If, however, it is desired to deviate from the writer's original proposals, then either Eq. (19) or (20) is more preferable than Eq. (17).

The author's test results are of great interest, as forming the first published experimental evidence with which to assess the accuracy of any theoretical approach. A program of such tests is under way at Manchester, but will not be completed for another year. The theoretical predictions obtained from Eq. (18) and (20) used in conjunction with the relevant design curves[15] are compared with Professor Bresler's test results in Table E. It is seen that there is fair agreement, and it is of interest to note that the scatter in the biaxially loaded column results is of the same order as that of the uniaxially loaded ones (as far as Eq.

(18) is concerned, at least) giving some indication that the method may be as accurate as the fundamental assumptions of limit design will permit.

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15. Pannell, F. N., "Failure Surfaces for Members in Compression and Biaxial Bending," submitted to ACI JOURNAL and pending publication.

AUTHOR'S CLOSURE

Professor Constantinescu-Catunesti's suggestion that the method can be extended for tensile as well as for compressive loads is an interesting one. Identification of Nikitin of ZNIPS as the original proponent of Eq. (4) is welcome, although unfortunately the original publication of Nikitin's work is not identified.

The derivation of Eq. (4), attributed to Nicula, is interesting, but may lead to an erroneous interpretation of this equation. The derivation proposed by the discusser appears to be based on the following assumptions:

1. Stresses are defined by conventional linear theory, superimposing effects of axial load and bending.

2. Effective cross-sectional properties such as area, section modulus, etc., are independent of the type of loading.

3. Failure under any loading condition is defined in terms of maximum stress on the concrete at the corner of the section.

Based on these assumptions Professor Constantinescu-Catunesti develops a linear equation similar to Eq. (4). However, it should be noted that the author's definitions of the loads P_x and P_y in the equation differ from those suggested by the discusser. As defined in the paper, the loads P_x and P_y are *ultimate* eccentric loads on the sections and thus cannot be obtained by a linear theory.

The misinterpretation which can result from Nicula's derivation is evident in the discusser's suggestion that quantities A, A', B, and B' are constants, leading to the statement that idealized curves for P_x and P_y are straight lines. In the author's formulation the quantities A, A', B, and B' would be variable coefficients and curves for P_x and P_y are not straight lines.

Mr. Gould focuses attention on an important aspect of structural design which requires knowledge of the mechanism of ultimate failure and of the nature of the ultimate loading conditions, which may not be proportional to the conventional design loading conditions.

Mr. Esty's discussion is particularly valuable as it presents some results of a large number of calculations on 13 typical sections with variable f_c', f_y, e_u, and E_c. Evaluation of Method A discloses the degree of approximation involved. On the average (for 215 cases) the error

of the method is 5.8 percent on the conservative side, and in extreme cases, the error calculated is of the order of 24 percent. Mr. Esty notes that the large deviations occur at small values of P, i.e., at high eccentricities. When Load P is small, failure is primarily due to flexure and axial load has little influence. For such cases use of Eq. (4) is not approprite. Equation for α to be used in conjunction with Eq. (7) is interesting, but it may not be generally valid, as pointed out by Mr. Esty.

Professor Pannell's discussion raises a number of questions. First, Professor Pannell is in error when he interprets the proposed Method A as a "plane approximation to a curved surface." As stated in the paper, the approximation involves "an infinite number of planes, each one applicable only for particular values of x and y, and thus, each plane defines only one point on the surface . . . ." Thus Eq. (4) defines a family of planes, each plane valid at one point only, and this is the essential reason for a fairly good approximation of the failure surface by this equation.

The other questions raised by Professor Pannell regarding Method A are the accuracy, simplicity, and general validity of the proposed Eq. (4). Undoubtedly Professor Pannell's proposed graphical solution for Load P_y is a more accurate method than that proposed by the author. This accuracy is obtained by incorporating a factor N, defined as a "deviation correction" factor. The value of N depends on geometric characteristics of the column cross-sectional dimensions, size and arrangement of reinforcing bars, properties of materials, and amount of concrete cover. Variation in N with aspect ratio b/d of rectangular section may be neglected, and values of N for certain classes of columns may be approximated by those calculated for a square column. However, values of N are significantly influenced by arrangement of steel reinforcement and separate charts must be used for columns with four bar and eight bar arrangement of steel reinforcement.* Variation in steel arrangement in adjacent faces again will require additional charts. Thus accurate solutions can be obtained only for those sections for which values of N are accurately known from available charts. Preparation of such charts for all possible types of columns would be a monumental task, if at all feasible. Whether practical designers would choose to rely entirely on availability of appropriate charts for accurate determination of column strength, or whether they would choose a more general although somewhat less accurate approximate method, would undoubtedly be decided on the basis of convenience. When charts are readily available for the case at hand, they would naturally be used. But when sections must be used for which charts for N values are not readily available, the designer needs some guid-

*Pannell, F. N., "Design of Bi-axially Loaded Columns by the Ultimate Load Method," *Concrete and Constructional Engineer* (London), V. 45, No. 11, Nov. 1960.

ance, and a generally applicable approximate method, such as proposed by the author, might be quite useful.

Professor Pannell states that the standard of accuracy of Eq. (4) is not acceptable. His evaluation is based on 24 cases listed in Table D. It is apparent from these calculations that the author's method is inaccurate for low values of P, i.e., for cases when eccentricities are high and failure is primarily due to bending moments and not due to axial load. Such cases are 3, 4, 7, 19, and 20, in Table D. Eliminating these cases from consideration, the maximum error for the remaining 19 cases is 12 percent, with an average error of 3.6 percent. In the author's opinion this standard of accuracy is acceptable, considering the complex nature of the problem. The five cases eliminated from consideration in Table D have values of (P/P_o) of 0.0281, 0.0405, 0.055, 0.0405, and 0.0276. The greater of these involves an axial load P of only 5.5 percent of P_o capacity of the column, and its design would be governed by flexure rather than by axial load. For such cases application of Eq. (4) is clearly inappropriate.

Professor Pannell questions simplicity of applying Eq. (4) to practical design cases. As the problem of combined action of axial load

TABLE F — COMPARISON OF COMPUTED VALUES OF P_i and P_u OF SECTION 12

Section	x	y	P_u	P_x	P_y	P_o	P_i	P_i/P_u	P_u/P_o	P_i/P_o
12-1	30.87	27.85	512	720	1320	5028	513	1.00	0.102	0.102
	16.12	14.05	1091	1575	2230		1130	1.04	0.217	0.225
	8.80	7.29	2019	2520	3300		2000	0.99	0.401	0.398
	6.12	2.37	2967	3100	4380		2855	0.96	0.590	0.576
12-3	33.37	32.21	524	715	1165	7548	482	0.92	0.069	0.064
	19.67	18.39	1028	1550	2445		1085	1.06	0.136	0.144
	10.77	9.47	2180	3070	4160		2300	1.06	0.289	0.305
	3.87	3.46	4919	5460	6120		4680	0.95	0.651	0.620
	1.41	1.35	6533	6720	6940		6220	0.95	0.865	0.825
12-5	21.71	38.39	424	850	600	4604	381	0.90	0.092	0.083
	10.11	19.67	1032	2120	1460		1061	1.03	0.224	0.231
	5.46	11.54	1916	2915	2370		1820	0.95	0.416	0.396
	3.35	6.86	2777	3550	3120		2680	0.96	0.604	0.582
	38.49	37.68	296	400	610		256	0.87	0.064	0.055
	15.52	13.31	998	1375	2150		1020	1.02	0.217	0.222
	10.11	8.19	1667	2120	2885		1658	0.99	0.362	0.360
	5.24	4.43	2753	3040	3590		2550	0.93	0.599	0.555
	1.29	1.36	4094	4170	4275		3900	0.95	0.889	0.848
	26.79	12.19	580	655	2270		570	0.98	0.126	0.125
	10.18	3.54	1966	2105	3775		1920	0.98	0.427	0.417
	2.13	1.16	3864	3900	4330		3700	0.96	0.840	0.805
12-7	31.80	33.40	409	560	850	7124	355	0.87	0.057	0.050
	19.81	19.14	822	1180	2020		830	1.01	0.115	0.116
	12.70	11.29	1621	2320	3510		1730	1.07	0.227	0.247
	8.40	7.10	2708	3480	4620		2745	1.01	0.380	0.385
	3.69	3.30	4783	5220	5825		4485	0.94	0.671	0.629
	1.49	1.49	6130	6270	6525		5820	0.97	0.860	0.817
	0.92	0.98	6520	6610	6760		6300	0.97	0.915	0.885

*For Section 12 see Fig. B.

with biaxial bending admittedly is a complex one, no solution can be as "simple" as the case of pure bending. Eq. (4) reduces the degree of complexity of this problem to that of singly-eccentric compressive load and thus may be considered a significant simplification. To the extent that formulas or charts are useful in solving the problem of a singly-eccentric compressive load, these are useful in obtaining a solution of Eq. (4).

Professor Pannell questions the validity of applying Eq. (4) to columns with unequal steel in adjacent faces. Only one such section (No. 12) has been considered in the group studied by Mr. Esty. Results of calculations for this section are shown in Table F, which indicates close agreement between the values of P_i and P_u. Again it is seen that relatively large errors occur when ratio P_u/P_o is less than 0.10. In four such cases the errors are 8, 10, 13, and 13 percent. Eliminating these cases, it is found that for the remaining 24 cases, with P_u/P_o greater than 0.10, the errors range from +7 to −7 percent with an average error of 1.5 percent. In the author's opinion this degree of accuracy is entirely acceptable, and thus Eq. (4) appears to be sufficiently accurate even for columns with unequal steel in adjacent faces.

The author is grateful to all discussers for their comments and criticism of his paper. These discussions served to define the degree of accuracy, the range of validity, and the proper use of the design criteria, and thus have materially contributed to the solution of the complex problem considered in the paper.

Discussion of a paper by G. E. Monfore and G. J. Verbeck:

Corrosion of Prestressed Wire in Concrete*

By WILLIAM E. DICKINSON, GEORGE D. RATLIFF, JR., M. UNZ, CARROL M. WAKEMAN, and AUTHORS

By WILLIAM E. DICKINSON†

The authors have added useful information to the rather limited data now in existence concerning the corrosion of prestressed wire in concrete. We agree generally with the conclusions of this report and are presently not recommending the use of calcium chloride in prestressed concrete.

Along with our general agreement we would like to make these comments on the conclusions:

Certainly in a research program such as this, it was logical to use increased amounts of calcium chloride, 3 to 4 percent. At the same time it would appear that more tests might have been made in the lower range of 1 to 2 percent calcium chloride by weight of cement, the amounts normally used. The conclusions also note that it has been suggested that there is a possibility of the existence of a threshold amount of calcium chloride below which significant corrosion does not occur. The comments on this suggestion seem to be well founded, and undoubtedly are the reasons for only a few tests at the lower amounts of calcium chloride.

The final disposition of calcium chloride in portland cement concrete is still not known; we are currently conducting basic research on this problem through our research fellowship at the National Bureau of Standards. A study of this type is involved and contains many of the unknowns that are still associated with the hydration of portland cement itself. Other studies now being made in England may shed more light on this entire subject.

The conclusion that the corrosion of prestressing wire is an erratic process and that quantitative evaluation is difficult appears to be worth repeating; without deferring interest in the possible effects of calcium chloride, it would appear that other factors may have more of an effect than has been studied and reported to date. In this study

*ACI JOURNAL, V. 32, No. 5, Nov. 1960 (*Proceedings* V. 57), p. 491. Disc. 57-24 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 12, June 1961 (*Proceedings* V. 57).

†President, Calcium Chloride Institute, Washington, D. C.

and as noted in the conclusions, voids were purposely built into the specimens and the corrosion was usually associated with these large voids adjacent to the wire; again this was a logical procedure in the study, but does not indicate what the results might be if the placement had been such as to eliminate voids. There are many operations in construction that could lead to difficulties if carried out improperly or contrary to known good practices.

Certainly this recommendation against use of calcium chloride in prestressed concrete need be directed to pretensioned members only. In post-tensioned members where the mass of the concrete is kept from contacting the wires by tubes, the use of calcium chloride in this concrete would be completely satisfactory. These comments are supported by portions of "Tentative Recommendations for Prestressed Concrete".* Section 302.2.4, states that "The use of calcium chloride or an admixture containing calcium chloride is not recommended where it may come in contact with prestressing steel," and Section 303.2.4, states that "Calcium chloride or an admixture containing calcium chloride is not recommended for use in grouting post-tensioned members."

Inasmuch as corrosion occurs at times in prestressed concrete wherein calcium chloride has not been used, we submit that other factors must play significant roles in this problem. This would indicate a need for more intensive research work on the part of the industry as a whole with the expressed purpose of determining the prime cause for this corrosion and developing a technique to prohibit its occurrence or, at least, eliminate its undesired effects.

The final and perhaps most important comment that we would like to make has nothing to do with this report as such, but it is so closely related that it may need clarification in the minds of some ACI readers. For use in concrete containing regular reinforcing steel, calcium chloride has had and still enjoys wide acceptance and use as the standard accelerator. In summary, we do not presently recommend the use of calcium chloride in pretensioned prestressed members, but we wholeheartedly endorse and recommend its use in regular reinforced concrete.

By GEORGE D. RATLIFF, JR.†

The authors deserve credit for a fine report and for again bringing to the attention of the profession the potential danger of chlorides in reinforced concrete. Although the deleterious effects of chloride salts have been mentioned in the literature as far back as 1913,[16] there is a tendency to forget. We have been studying some of the problems of corrosion of steel in concrete containing chlorides for many years

*ACI-ASCE Committee 323, "Tentative Recommendations for Prestressed Concrete," ACI JOURNAL, V. 29, No. 7, Jan. 1958 (*Proceedings* V. 54), pp. 545-578

†Project Engineer, Granco Steel Products Co., St. Louis, Mo.

TABLE A — RESULTS OF 28 DAY CORROSION TESTS OF GALVANIZED AND UNCOATED STEEL*

Storage condition for concrete	Depth of embedment, in.	Area corroded, percent	
		Galvanized	Uncoated
150 F, 10 percent relative humidity	½	20	80
	1	52	100
	3	98	100
100 F, 90 percent relative humidity	½	T†	35
	1	2	25
	3	T†	15
85 F, 40 percent relative humidity	½	2	8
	1	T†	2
	3	T†	5

*Embedded in separate concrete cylinders containing 1 percent calcium chloride by weight of cement.
†Trace

and have found the subject to be complex and the basic theory controversial.

The authors state in their conclusion that corrosion "is not greatly influenced by curing temperatures." In our work we have found instances where there is a pronounced temperature effect. In one study, 1 x 4 x 20 in. gage strips of both coated and uncoated steel respectively were embedded in 6 x 12 in. concrete cylinders. The concrete contained 1 percent calcium chloride by weight of cement and had a slump of 7½ in. After curing for 1 day in the cylinder molds, the cylinders were stripped and placed in the environment indicated in Table A.

The cylinders were broken at 28 days. The area of metal which had corroded is shown in Table A. Duplicate specimens were tested and average values are given. In every case the results were in close agreement. The results indicate a marked acceleration of the corrosion rate at higher temperatures. It is also apparent that galvanized steel is more resistant to corrosion than uncoated steel at ordinary temperatures.

Akimov[17] suggests that the corrosion rate of iron may be the result of many different functions of temperature. For example, if oxygen solubility governs the speed of the reaction, since the solubility decreases as the temperature increases, the corrosion rate may be slowed at higher temperatures for lack of oxygen. However, in an oxygen-rich environment, the corrosion reaction usually increases exponentially with temperature.

In this study an attempt was also made to determine the effects of cover. Three identical strips of steel were embedded in the same cylinder. The strips were embedded with nominal covers of ½, 1, and 3 in. In Table A, the effect of depth of cover is most apparent for the uncoated steel at 100 F, showing increasing corrosion with a decrease in cover. For the galvanized steel at 150 F we found that the effect

of cover was opposite to what we expected and what is described in the literature. The zinc-coated strips with the most cover showed the most corrosion.

We are particularly interested in the authors' finding that the corrosion of embedded wires is accelerated when brass or stainless steel forms are used, and the corrosion of the wires is inhibited when steel forms are used. Building slabs frequently contain copper pipes, galvanized conduit, and other non-ferrous metals. Metals with a higher electromotive force than iron, such as aluminum, magnesium, and zinc, tend to corrode sacrificially and protect the iron. Metals below iron in the electromotive series, such as copper, brass, or alloy steels, are themselves protected by the corrosion of the iron.

We have had some test experience with galvanized steel forms. These forms were used to form the bottom of 9 x 15 x 3½-in. slabs. Small steel rods were embedded in the concrete and welded to the galvanized forms, thus insuring good electrical contact. The calcium chloride content was varied from 0.05 to 0.5 percent by weight of the cement. Some of the slabs were cured at room temperature and some in a 150 F oven. At 28 days they were opened for inspection. When corrosion occurred the galvanized forms showed corrosion and the embedded rods did not. This indicates that the galvanized forms became anodic to the embedded steel reinforcing. When corrosive conditions are present, such forms will corrode to protect the reinforcing steel.

The authors suggested the possibility of a threshold chloride limit necessary for corrosion. The data in Fig. A were developed from the previously mentioned test. The area of form corrosion was plotted against the calcium chloride concentration. From the figure it is apparent that the threshold calcium chloride content necessary to start corrosion in this type of test is about 0.08 percent by weight of cement, when the specimens are held at 150 F. At room temperature, the threshold is well below 0.2 percent, possibly about 0.1 percent calcium chloride by weight of cement. Since calcium chloride is 64 percent chloride, the chloride threshold is 0.64 times the calcium chloride threshold.

It is important to include chlorides from all sources in evaluation of the corrosion potential of a batch of concrete. Chlorides may be present in the mixing water, as sodium chloride impurities in the aggregate pile, and in many proprietary admixtures, as well as in straight calcium chloride additions.

Chloride salts are dangerous when present in the concrete, and also when they are used in contact with concrete, as shown by Baumel and Engell,[18] Lewis and Copenhagen,[19] and others. In some tests which we ran, a measured application of calcium chloride was placed on top of a slab and the surface kept wet. The steel in the slab had a cover

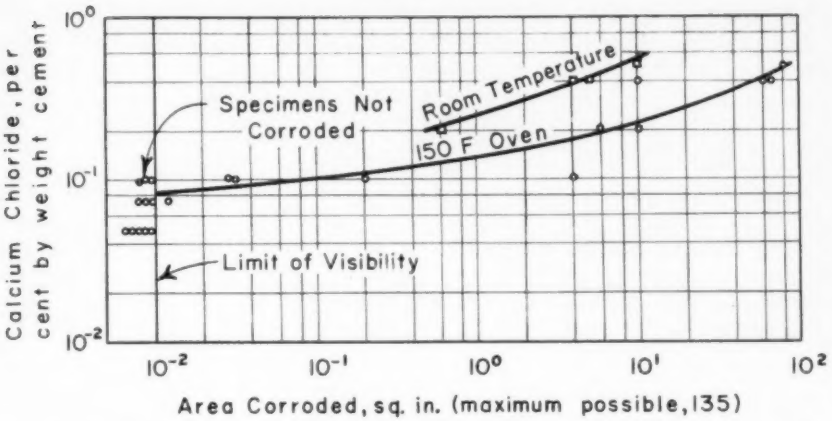

Fig. A—Calcium chloride concentration versus galvanic corrosion of galvanized steel forms for reinforced concrete slabs

of 1½ in. of concrete, which had no visible cracks. At the end of 18 months the chloride ions had migrated through the 1½ in. concrete cover and were attacking the steel in the vicinity of the calcium chloride application.

Since it has been shown that the chloride ions can penetrate into concrete to attack the reinforcing steel, the use of salt on bridges or parking structures should be given a new look. The development of a noncorrosive de-icing agent or a protective sealant which will stop the penetration of chlorides is greatly needed at this time.

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By M. UNZ*

Prestressed concrete has normally been found to be of remarkable anticorrosive properties.

An intensive positive aging process apears to be responsible for this splendid performance. A self-sealing effect develops under influence

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of incipient corrosion currents, which raises the insulation resistance to a multiple of its original value. This process has been observed on many miles of buried prestressed concrete pipe and experimentally confirmed.* The effect is of particular importance in anodic areas where an impermeable, protective "cement skin" is created by the iron oxide products of the initial corrosion process.

This fact may serve to explain why corrosion breakdowns often occur in the first year after the laying of a line. In the early stages of the natural sealing process the steel wire is badly exposed to the corrosive effects of the environments. The corrosion currents subside only after the build-up of a cement skin of adequate strength, until a state of safe equilibrium is reached.

In this regard it would be of great assistance if the authors could answer the following questions:

1. Has cement slurry been applied to the steel surface, for improving the anticorrosive properties?

2. Has artificial aging using small anodic currents been attempted?

3. What other means could be suggested to support the formation of the cement skin?

We are indebted to the authors for the comprehensive data published in the paper. The wide scope of evidence presented, constitutes an important steps towards an inherently safe concrete structure.

By CARROL M. WAKEMAN†

The authors' paper is both timely and valuable in this age of prestressed concrete. It can be particularly useful in the consideration of reinforced concrete in the coastal areas where chlorides may diffuse toward the steel from air-borne salts or capillary action of sea water.

Designers of prestressed units for use in marine construction should take particular cognizance of the authors' Conclusions 3-j and 3-h, p. 514 ". . . corrosion . . . may occur whether the wire is or is not subject to external load." We frequently hear the opinion expressed that prestressing concrete endows it with increased durability in sea water, as compared to an identical unit which is not prestressed. The reason behind such hypothesis is that the concrete is placed in compression which, by some magic, decreases permeability, porosity, or both. In the opinion of the writer, nothing could be further from the truth. If the authors' Conclusion 3-h is valid, then it behooves the harbor engineer

*See Unz, M., *Corrosion*; V. 12, No. 10, Oct. 1956, p. 526; V. 16, No. 6, June 1960, p. 289; V. 16, No. 7, July 1960, p. 343. Also *Technical Paper* No. 18, U.S. Bureau of Standards, 1913.

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to design prestressed units with a minimum of 3 in. of good concrete cover* which is widely accepted as a standard for reinforced concrete in a marine environment.

The writer wishes to cite another example of stress corrosion which occurred in a high strength prestressed wire reinforcement for a large concrete cast-in-place cylindrical storage tank used for storing lime slurry. The tank was one of several built recently in the San Francisco area on the waterfront. It was reinforced externally by a spiral cold-rolled wire cage, tensioned by passing a larger wire through a traveling orifice around the outside of the tank. Metal connectors were used to join the ends of each reel of wire. These connectors were cathodic to the wire. The tensioning wire was later covered with about 1 in. of shotcrete. About 3 years after the tanks were placed in service, the slurry tank burst. The investigation which followed indicated that the tensioning wire had corroded to the point that it could no longer resist the outward stresses of the full tank of slurry.† An examination of the shotcrete cover indicated a porous structure which contained chlorides. The latter presumably had originated from wind-blown spume from San Francisco Bay and had permeated through the interior mass of the shotcrete to the tensioning steel. Thereafter corrosion progressed as so clearly described by the authors.

Fragments of the corroded wire were subjected to a metallurgical examination. These tests revealed extensive laminar longitudinal cracking in the stressed wire, in addition to severe corrosion damage (see Fig. B, C, and D). Long pieces had flashed from the wire, producing section losses approaching 40 percent.

The report of the metallurgist concluded that:

1. Laminations had occurred in the wire at the time of the first drawing, with perhaps small cracks being formed along these laminations.

*Wakeman, C. M.; Dockweiler, E. V.; Stover, H. E.; and Whiteneck, L. L., "Use of Concrete in Marine Environments," ACI JOURNAL, V. 29, No. 10, Apr. 1958 (*Proceedings* V. 54), pp. 841-856.

†Unpublished report 1959, by C. M. Wakeman.

Fig. B — Photomacrograph (magnification 8×) showing a specimen of wire notched with a file and broken by bending to expose a longitudinal crack formed by a lamination. The dark area indicates prior corrosion in contrast to the remainder of the section which was broken for the photograph

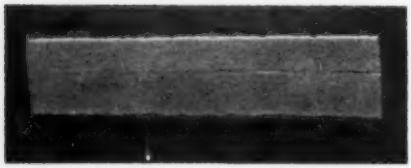

Fig. C—Photomacrograph (magnification 7×) showing a portion of the same wire as in Fig. 1. It has been ground, polished and etched to show the longitudinal lamination. Deep and severe corrosion has taken place along the crack. Numerous inclusions can be seen in this specimen under the microscope

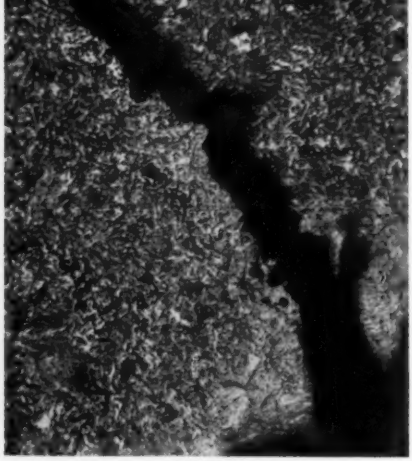

Fig. D — Photomicrograph (magnification 400×) showing the grain structure of a section of the wire in the laminated area. (The lamination runs diagonally across the photograph). The grain structure is normal, indicating that the crack resulted from stresses

2. During prestressing, these cracks developed to a point where they either broke out during prestressing or after the shotcrete had been placed, forming flat areas.

3. Prestressing to 60 percent of the original tensile strength caused additional stresses in the wire.

4. Corrosion was accelerated at these areas of high stress concentration. Examination of the corroded areas indicated the corrosion was of a chemical nature.

In addition to the general corrosion of the stress wire, caused by chloride penetration, there were other factors which contributed to its failure. The stress wire was furnished in 250-lb rolls. Specifications provided that the tank be wound continuously. This necessitated the use of wire connectors to join the successive reels. These connectors were made of galvanized steel and were cathodic to the stress wires. In other words, the stress system was, in effect, composed of a number of electrical cells, wherein corrosion of the wires at anodic areas near the junction of the dissimilar metals was accelerated.

Conclusion 1 of the metallurgist, i.e., that the laminations in the wire probably occurred at the place of manufacture, should point out the necessity for every user of cold-rolled steel to specify a finished product which is free of such flaws. This, of course, implies adequate job inspection.

AUTHORS' CLOSURE

As Mr. Dickinson notes, we purposely used 4 percent calcium chloride in many of our studies because we were seeking the effect of factors in addition to calcium chloride. However, we did include numerous tests using lesser amounts of calcium chloride. For instance, the study reported in Table 2 included equal numbers of specimens containing 0, 2, and 4 percent calcium chloride. Also, the investigation of chlorides in water-cement slurries included amounts of calcium chloride as low as 0.25 percent. In this connection, the discussion by Mr. Ratliff should be noted in which calcium chloride concentrations as low as 0.05 percent were studied. Mr. Dickinson implies that voids were purposely introduced in most of our specimens, but actually the number of such specimens was only 24 out of 128 specimens included in the investigations.

The data presented by Mr. Ratliff concerning the effect on corrosion of storage at various temperatures is interesting and in the direction that we would expect. In our tests of the effect of curing temperature, specimens were steam cured at 140 or 180 F for 16 or 12 hr and were then stored at room temperature for periods generally from 1 month to 2 years. Such specimens were compared to other specimens which were cured for 1 or 3 days at normal room temperature before being stored for the various periods at room temperature. The corrosion observed after extended periods of storage at room temperature was not greatly affected by the steam curing. We would expect that an increase in storage temperature would result in an increase in corrosion, as Mr. Ratliff observed. The observation concerning the migration of chloride ions through concrete is timely and has important implications regarding the use of corrosive de-icers on auto traffic surfaces, particularly where insufficient cover over reinforcement or concrete of high permeability is involved.

Dr. Unz inquires as to tests of cement slurry coatings on prestressing wires. We did undertake a limited number of such tests and did observe an improvement of anticorrosive properties. The only tests in which we applied external voltages were tests in which the wires were immersed in solutions. Of course, small currents are involved in the study where other metals in addition to the steel wire are in contact with the concrete. Our investigations indicated that a most important factor in preventing corrosion is to provide good bonding of a dense concrete to the wire by adequate placement procedures, which is in agreement with Dr. Unz's observations.

Mr. Wakeman has made an important contribution by calling attention to the inherent danger to coastal prestressed concrete structures due to air-borne chlorides from sea water.

In conclusion, the authors thank those who have taken time to prepare these discussions in which additional information has been made available and certain dangers have been pointed out.

Discussion of a paper by L. H. Kristof:

Monolithic Cast-In-Place Concrete Pipe*

By G. E. TROXELL, C. T. McNEILL

By G. E. TROXELL†

Although cast-in-place concrete pipe has been extensively used, some projects have not been as satisfactory as was intended. This has resulted from improper practices in the field rather than from any defects in the general construction principle involved. The two field practices which have led to unsatisfactory results are: (1) lack of proper curing and (2) premature loading of "green" pipe.

On one large irrigation project, several thousand lineal feet of 30, 36, and 42 in. diameter cast-in-place, one-stage, machine method pipe were found to have developed longitudinal cracks along the center of the crown and invert. Several circumferential cracks were also observed. A study of the construction methods and schedules used on this job disclosed the following factors which contributed to the cracking:

1. A concrete mix containing 6 sacks of cement per cu yd had been used although later operations showed that a 5-sack low-slump mix gave satisfactory strengths and appreciably reduced the potential shrinkage of the concrete.

2. A natural current of warm dry air was permitted to flow through the new pipe line. The resultant drying of the inner surface of the pipe while the outer portions remained damp caused high shrinkage stresses, both circumferentially and longitudinally. In later constructions this condition was avoided by sealing open ends of the line to retain the moisture in the concrete, and by flooding the line as early as possible, preferably not later than 2 weeks after casting.

3. The inner forms and supports were removed after 1 day and backfilling over the pipe was completed within 4 days, while the concrete was still relatively weak. This loading caused high circumferential tensile stresses at the middle of the crown and the invert. These stresses, in combination with the high drying shrinkage stresses, were the cause of the cracking along crown and invert.

*ACI Journal, V. 32, No. 5, Nov. 1960 (*Proceedings* V. 57), p. 533. Disc. 57-26 is a part of copyrighted Journal of the American Concrete Institute, V. 32, No. 12, June 1961 (*Proceedings* V. 57).

†Member American Concrete Institute, Professor of Civil Engineering, University of California, Berkeley.

In later constructions the inner forms and supports were kept in place for 2 days and the major backfill, exclusive of a light initial cover for curing purposes, was deferred for a minimum of 10 days, and all heavy traffic was kept off for at least 28 days. The revised curing and construction operations resulted in a superior pipe line free from cracking.

Repairs to the cracked pipe were accomplished with epoxy resins. The longitudinal cracks were repaired by a coating about ¼ in. thick which was feathered out at the edges to give a hard strip coating about 3 in. wide. At the circumferential cracks a plasticized epoxy resin was applied over a 2-in. width and with a maximum thickness of ⅜ in. It remained rubberlike and should allow movement at a joint without rupturing the resin or causing new cracks in the pipe at the edges of the band of resin.

By C. T. McNEILL*

The writer has been associated with the design and placing of irrigation systems of cast-in-place pipe and would like to comment on developments, particularly with reference to the one-stage process.

The statement is made that "Reinforcing steel should be used in the pipe wall when utilizing this conduit under roadways, streets, and other critical locations." Assuming good concrete with compressive strength of 3000 psi, or better, experience has shown that monolithic concrete pipe, with 2 ft of earth cover, can safely be used for road crossings and also for locations paralleling roads subject to general traffic. The Salt River Project, near Phoenix, Ariz., has many such installations with no structural failures. A structural analysis made at this project showed that cast-in-place unreinforced concrete pipe is capable of withstanding loads in excess of H-20 surface loads with an impact factor of 2.0 transmitted through 2 ft of compacted soil (unit weight equal to 110 lb per cu ft). There are many other cast-in-place pipe installations in California under city streets and county roads without any failures to date.

It is stated that the density of the pipe wall is often considerably less than in precast concrete pipe due to limitations of vibrations during placement. This is undoubtedly true for monolithic processes which do not employ vibration in concrete placement. However, there are processes where the concrete is consolidated by vibrators attached to the machine in much the same manner as in precast vibrocast pipe. A general view of construction of a 48 in. cast-in-place concrete pipe on a curve is shown in Fig. A.

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The statement is made that the maximum safe backfill depth over unreinforced pipe that customarily is used for average conditions is about 5 ft. Experience, tests, and computations have shown, however, that high quality cast-in-place concrete pipe lines can safely carry fills greatly in excess of 5 ft. For instance, a cast-in-place 60-in. test line in Yuba City, Calif., withstood a test load, of 14,254 lb per lineal ft without any cracking, which is equivalent to a fill of over 30 ft of saturated top soil weighing 110 lb per cu ft. Likewise the test load of 25,090 lb per lineal foot which a 120-in. test section of monolithic pipe carried without visible cracking, is equivalent to a height of fill above the top of the pipe of 23.4 ft for saturated top soil weighing 110 lb per cu ft.

Fig. A—Construction of a 48 in. cast-in-place concrete pipe on a curve

The question is raised as to the feasibility of using monolithic pipe under continuous flow conditions. In the past this type of pipe was used primarily where the flow was intermittent. In recent years, however, irrigation districts, have constructed, or are in the process of constructing, monolithic pipe lines as part of the main lateral system. Fig. B shows a view of a 30 in. diameter cast-in-place concrete pipe suspended for about 45 ft. The pipe was constructed in August 1952 and the soil was eroded during the floods of December 1955, near Westley, Calif. The photo was taken in March 1956.

The duties of the man who works inside the machine are described on p. 545. It should be understood that this workman does not fasten the forms together as stated. This operation is performed by the workman who feeds the forms into the machine. It should also be noted that air circulation and cooling inside the machine can easily be accomplished by an electrically operated fan. Fig. C shows a view of an electrically operated machine for constructing a 60 in. diameter cast-in-place concrete pipe. The machine moves to the right, extruding pipe to the left.

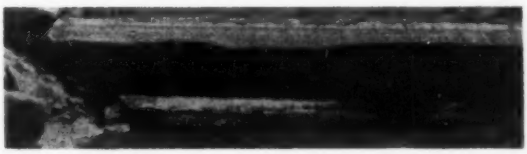

Fig. B — View of a 30 in. diameter cast-in-place concrete pipe suspended about 30 ft

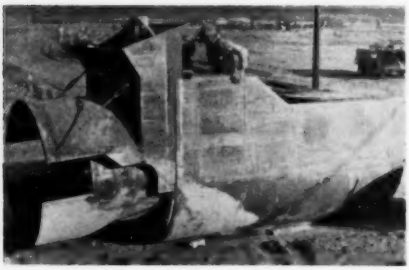

Fig. C—Electrically operated machine for constructing a 60 in. diameter cast-in-place concrete pipe

Possible adverse effects of vibration are discussed by the author. It must be borne in mind that proper controls are necessary to make quality pipe, either precast or monolithic. The tendency for freshly placed concrete to slide off the top portion of the oiled metals forms in one-stage construction can be controlled by use of well graded concrete of the proper consistency and by care in preventing overvibration.

The problems encountered with excessively loose soils are also discussed. One of the functions of the long heavy steel sled which rides on the bottom of the trench in some one-stage processes is to smooth out minor irregularities which might result from loose soil. Concrete is placed immediately behind the sled. The machine, in fact, is designed to exclude soil from the space to be occupied by the concrete. Excessively loose, cohesionless, dry sand is admittedly difficult to control. In the few places where such a condition exists the soil should be consolidated to provide a stable foundation.

The answer to the problem of "pitching" of the pipe making machine is to provide a reasonably good vertical alignment of the trench bottom. This is generally possible. If sharp changes in vertical alignment are necessary, a precast angle fitting may be used at the angle joint (which would also be required in a precast line if the pull in the joint exceeded the allowable tolerance).

Discussions of a paper by Lyle E. Young:

Simplifying Ultimate Flexural Theory by Maximizing the Moment of the Stress Block*

By IQBAL ALI, GEOFFREY BROCK, LADISLAV B. KRIZ, and A. SENI

By IQBAL ALI†

The stress functions for concrete in direct compression proposed by Professor Young are ingenious. The last two are simple and particularly amenable to convenient analytical treatment.

The concept of identifying the ultimate moment carrying capacity of a flexural member with the maximum resistance attained by the stress block developing in the form of the stress function in direct compression, however, is not entirely new. The writer outlined such an approach,[5] using the exponential type stress function suggested in an earlier paper by the author.[6] Several such studies have since been presented.[7,8,9]

The author has derived the shape of the stress block corresponding to the ultimate load, by maximizing its moment about the neutral axis, rather than the centroid of the tensile reinforcement. The reason for this choice is not clear, unless it was meant to attain simplicity at the expense of realism. The shape of the stress block corresponding to the ultimate moment carrying capacity of a beam section, as obtained by maximizing the internal resisting moment, is by no means invariable, even for the same quality of concrete. It varies considerably, in respect of the limiting strain, depending not only on the shape of the beam cross section, but also on the percentage of tensile reinforcement and its stress-strain characteristics. Using the second stress function proposed by the author, it may be shown,[5] that for a rectangular section with the tensile reinforcement exhibiting a linear elastic response and a flat yield, values of the relative limiting strain $\varepsilon_u/\varepsilon_o$ corresponding to the maximum internal moment, range approximately from 1.4 to 2.0. Both under- and over-reinforced sections tend to mobilize their maximum flexural resistance at lower levels of the limiting strain, while higher values are attained when the reinforcement is more nearly balanced.

The author's solution corresponds to a hypothetical case with infinite reinforcement, where the neutral axis lies at the tensile reinforce-

*ACI JOURNAL, V. 32, No. 5, Nov. 1960 (*Proceedings* V. 57), p. 549. Disc. 57-27 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 12, June 1961 (*Proceedings* V. 57).

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ment level. For other cases the solution does not truly represent the stage at which the maximum internal moment of the beam section is mobilized, but corresponds to lower moment levels. The rather close check between calculated and observed values of the ultimate moment as shown in Table 1 may be explained by the fact, that the internal moment is rather insensitive to variation in the limiting concrete strain. True maximum values would be slightly higher than those calculated.

The analysis developed is no doubt relatively simple and appears adequate for practical purposes, despite the rather arbitrary procedure of maximizing the stress block moment about the neutral axis and identifying this stage with the maximum moment carrying capacity of the section as a whole.

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GEOFFREY BROCK*

The method proposed by Professor Young has already been considered in a classic paper on plastic design of reinforced concrete.[10] Although it had not then been shown that the stress-strain curves for concrete in flexure and direct compression were similar, Whitney used typical stress-strain curves from cylinders to construct curves of moment of resistance against fiber strain. He wrote "The maximum ordinates [in Fig. 3(a)] give the maximum resisting moments in the beam. They are significant quantitatively only in a general way but are of great value in indicating what is happening in the beam. The actual maximum value can best be determined from tests on beams."

Professor Young claims that we should not rely on beam tests in this way. In dismissing them as empirical he neglects the underlying empiricism of his alternative. This is implicit in the use of typical stress-strain curves. Whether we use the ingenious experiments of Hognestad, Hanson, and McHenry,[1] the interesting modification recently described

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by Smith,[11] or controlled cylinder tests we lean heavily on experiment. And experiments to obtain complete stress-strain curves must always be rather esoteric and dependent on experimental technique.[9] By assigning analytic expressions to such curves the author has tended to invest them with a degree of generality which they do not really merit.

While any correlation with stress-strain curves is of considerable interest, it does still appear that the most reliable information on beam strengths will be obtained from beam tests.

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By LADISLAV B. KRIZ*

The author assumes that at ultimate strength of a reinforced concrete beam, the moment of the resultant compressive force about the neutral axis is maximum. However, the resisting moment of a beam (i.e., the moment of the resultant compressive force about the centroid of the tensile reinforcement) is not proportional to the moment of the compressive force about the neutral axis.[7,8] Moreover, the position of the neutral axis is not constant during bending. These circumstances were neglected by the author when he maximized the moment of the compressive force about the neutral axis.

In maximizing the moment y_oC, the author differentiates the three equations (1a), (1b), and (1c) with respect to ε_o. According to Fig. 2 and the Functions 1, 2, and 3, the strain ε_o corresponds to the maximum concrete stress, f_c'. Thus, the strain ε_o is a constant depending on the stress-strain curve in question, and cannot be considered as a variable in Eq. (1a), (1b), and (1c). The differentiation of these three equations should have been performed with respect to the variable strain in the extreme fibers, i.e., with respect to ε_u according to the notation used in those equations. For example, differentiating Eq. (1a) which should read:

$$y_oC = b(kd)^2\left[E_T\left(\frac{\varepsilon_u^3}{5\varepsilon_o^2} - \frac{\varepsilon_u^2}{2\varepsilon_o} + \frac{\varepsilon_u}{3}\right) - f_c'\left(\frac{2\varepsilon_u^3}{5\varepsilon_o^2} - \frac{3\varepsilon_u^2}{4\varepsilon_o^2}\right)\right]$$

with respect to ε_u (assuming k = constant) and equating to zero yields:

$$\frac{d}{d\varepsilon_u}(y_oC) = b(kd)^2\left[E_T\left(\frac{3\varepsilon_u^2}{5\varepsilon_o^2} - \frac{\varepsilon_u}{\varepsilon_o} + \frac{1}{3}\right) - f_c'\left(\frac{6\varepsilon_u^2}{5\varepsilon_o^2} - \frac{3\varepsilon_u}{2\varepsilon_o^2}\right)\right] = 0$$

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The solution of this equation is $\varepsilon_o/\varepsilon_u = F(\varepsilon_o, E_T, f_c')$
Specifically, for $E_T = 1000\ f_c'$, and $\varepsilon_o = 0.002$, $\varepsilon_o/\varepsilon_u = 0.75$.

The value of 0.75 can be compared with the value of 0.80 obtained by the author in spite of several mistakes in the arithmetic. It appears that the functions (1a), (1b), and (1c) are of such a nature that considering either ε_o or ε_u as the variable yields similar values of the strain ratio $\varepsilon_o/\varepsilon_u$, for which the moment y_oC is maximum. This similarity, however, in no way justifies the procedure of the author.

The value of the theoretical ultimate strain (within certain limits) has little influence on the calculated value of the ultimate moment, as the strains in the extreme fibers increase rapidly with a small increase in the bending moment when the ultimate strength is approached. Therefore it is understandable that the agreement between the calculated and test moments was satisfactory in spite of the manner in which the ultimate strains were derived. Thus Professor Young's paper points out again that the exact shape of the stress block and the value of the theoretical ultimate strain are relatively unimportant and that good results can be obtained by using reasonable approximations, such as a rectangular stress distribution.

By ALFIO SENI*

The paper contains a method for the calculation of the ultimate moment of a reinforced concrete beam which the author implies simplifies ultimate flexural analysis.

Although of theoretical interest, the design of beams by this method is more complicated than by other methods which use a rectangular shape for the stress block, instead of one of the functions proposed by the author.

The Whitney method, for instance, which uses a rectangular shape of stress block, gives for the ultimate moment a simpler formula and does not take into account the ultimate strain of the concrete, ε_o, which is usually unknown.

For instance, applying the Whitney method to the example given by the author, we find

$$m = \frac{40{,}000}{0.85 \times 4000} = 11.8 \quad \text{and} \quad a = p\,m\,d = 0.017 \times 11.8 \times 6 = 1.2$$

Now since $p < 0.537/m = 0.0455$, the failure occurs with the steel and the moment is

$$M = 0.85 \times 4000 \times 1.2 \times 4 \times \left(6 - \frac{1.2}{2}\right) = 88{,}300 \text{ in.-lb}$$

As can be seen the difference between this result and that of 89,200 in.-lb as found by the author is only about 1 percent.

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In regard to the accuracy of both methods one can see from Table 1 that the author's method gives percentages of error which are at least equal to those of Whitney's, so that since the latter is simpler it is preferable.

It is interesting to note that Russian specifications for reinforced concrete have used ultimate strength design since 1938 including the same rectangular stress block diagram. Furthermore, the Soviet Union and some of the other eastern European nations are the only countries which have completely abandoned the elastic design method for reinforced concrete.

They adopted this shape because numerous tests showed that it was the most applicable shape of the many studied, being both accurate and simple.

Because of the simplicity of the formulas it was easy to establish design tables, which give the required height d knowing the reinforcement percentage, or alternately to find the design requirements when different values of f_c', f_y, and the respective safety factor are prescribed.

These tables have been in general use for many years and their soundness is demonstrated by the countless well-designed structures in use today.

Discussion of a report by ACI-ASCE Committee 333:

Tentative Recommendations for Design of Composite Beams and Girders for Buildings*

By SEPP FIRNKAS, A. ZASLAVSKY, and COMMITTEE

By SEPP FIRNKAS†

The recommendations of ACI-ASCE Committee 333 close an important gap and will be highly appreciated by many consulting engineers and architects. However, the writer would like to help clarify some aspects of the composite construction and help to further advance the applications of this highly economical structural system.

Section 107 — Deflections

Deflections are quite frequently used as criteria for the acceptability of a structural member. Therefore, the designer must be fully aware of the construction problems involved and should elaborate and specify an erection and placing schedule, since these factors have considerable influence on the deflection of the composite beam. To deal summarily with deflections as it is done in Section 107, is to neglect the different properties of composite beams and the different materials used to create these beams. A steel-concrete beam will show different long-time deflection characteristics than a prestressed and cast-in-place beam. To use one half the value of the modulus of elasticity, E_c, for the transformed section for dead load deflections may be an approximation for certain cases, but certainly cannot generally be used. Following the theory of differential shrinkage and creep three possibilities, viz., (a) tension, (b) compression or (c) a neutral state can develop, caused by shrinkage of cast-in-place concrete, and shrinkage and creep of prestressed concrete. Deflections downward or upward result, depending on the stress distribution over the section. It appears, therefore, that the estimate of deflections should be treated individually in full consideration of the materials used and the actual stress distribution over the composite section.

Section 108 — Continuity

In Section 108.2.2 if a preformed joint over the support of continuous beams (negative moment assigned to prefabricated concrete or steel section) is provided, special reinforcement, nevertheless, should be placed next to the joint to prevent cracking to the left or right of the joint. Experience shows that deformations away from the support, es-

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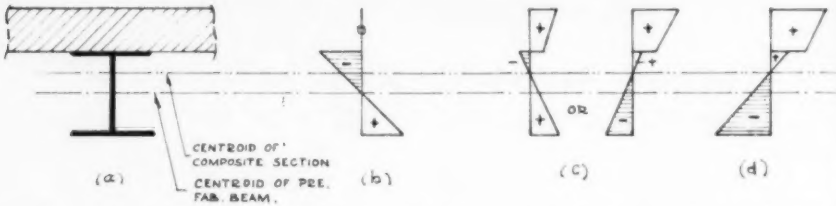

Fig. A — (a) Composite section, (b) beam prestressed (upward deflection energy stored in beam), (c) shores released dead weight and released energy act on composite section, (d) DL + LL act on composite section

pecially for industrial floor live loads, are of such magnitude as to create cracks at some distance from the preformed joint.

Section 202.3—Deformational stresses

An attempt should be made to evaluate deformational stresses wherever prestressed concrete is used for the composite section.

Section 203 — Determination of ultimate flexural stresses

Note that effective flange width is only six times the slab thickness following the "Building Code Requirements for Reinforced Concrete (ACI 318-56)," * as compared to eight times the slab thickness as mentioned in Section 105.2.

General considerations

The structural and economic advantages of this construction method are self-evident and need hardly be explained. But one additional possibility should have been mentioned in this explanation: prestressing through shoring. Under the heading Deflection it is mentioned that an additional effective load produced by the energy stored in the beam, if the shores are driven too tight, will produce additional deflection. I cannot agree with this statement. An upward deflection will cause prestress in the beam. The energy thus stored is just the energy that is required to deflect the beam upward. At the time of unshoring, the composite section with a considerable higher moment of inertia is realized, and the stored energy will act on this section, and force only a small amount of downward deflection as compared to the original upward deflection.

A qualitative evaluation (see Fig. A) of the stresses shows the considerable advantage of "prestressing" the beam by upward deflection. Furthermore, this seems to be a logical way to keep working stresses of beams within allowable limits. Screeding to equal slab thickness poses no real problem if screed chairs are used which in turn are supported on the beams.

*ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-56)," ACI JOURNAL, V. 27, No. 9, May 1956 (*Proceedings* V. 52), pp. 913-986.

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By ARON ZASLAVSKY*

The recommendations are a helpful guide for the design practice of composite sections and it is hoped that a final report will follow in the near future.

The following remarks are confined to Chapter 4 dealing with slabs on steel beams.

Section 402.2 states that it may be assumed that all dead and live loads are resisted by the composite section, even when no temporary supports are used. This is, of course, a departure from the conventional elastic working stress method. The writer feels that this departure has been long overdue, particularly when no proportionality exists between stress and load (which is the case in composite sections).[14,15]

But since the recommendation is actually justified by the ultimate (critical) plastic resistance, the writer sees no reason why a clear-cut ultimate strength theory representing a real physical stage should not be used, with due allowance for the usual limitations regarding deflections, cracks, etc., at the working level. The method proposed in Section 402.2 is in fact a numerical compromise which leads to a safe (or rather too safe) design but does not reflect the actual physical behavior of the composite beam. A code of practice should attempt to provide a formula giving not only safe numerical results but also a more or less realistic picture of the behavior of the structure and a rational design method.

As an example consider a composite section consisting of an unsupported built-up steel girder and a reinforced concrete slab as shown in Fig. B (dimensions given in centimeters). The steel beam is subjected to a given bending moment $M_D = 100$ ton-meters due to dead weight. The yield stress is $f_y = 2400$ kg per sq cm and for simplicity assume the safety factor = load factor to be $K = 2$, i.e., the allowable working stress equals $f_s = 1200$ kg per sq cm. The moment of inertia of the steel

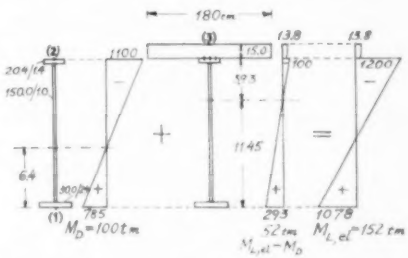

Fig. B — Superposition of stresses in the working stage

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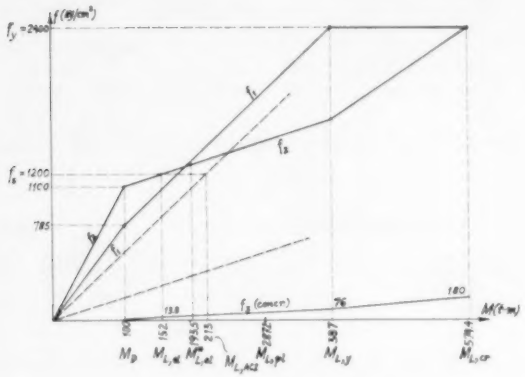

Fig. C — "Loading history" of section

section equals $I_s = 813{,}000$ cm^4 and the moment of inertia of the composite transformed section ($n = E_s/E_c = 10$) equals $I = 2{,}046{,}000$ cm^4.

Fig. C shows the "loading history" for the different loading stages. The stresses f_1 (bottom steel fiber), f_2 (top steel fiber) and f_3 (top concrete fiber) are plotted against the bending moment produced by the dead load, M_D, and the total dead + live load, M_L; M_L also includes additional dead weight resisted by the composite section. (Transition curves were omitted in Fig. C)

By the conventional elastic (working stress) method the total allowable moment, producing $f_2 = f_s = 1200$ kg per sq cm at the most stressed steel fiber, equals $M_{L.el} = 152$ ton-meters (see Fig. B and C). It should be noted that the rate of increase of stress f_2 is greater than that of f_1 in the dead load stage, but in the live load stage (composite section) the picture is reversed as shown in Fig. B and C. For this reason Fiber 1 is the first to yield ($M_{L.y} = 387$ ton-meters). While the appearance of the working stress is governed by Fiber 2, the appearance of the first yield is governed by Fiber 1. (This depends mainly on the ratio between the areas of the flanges. Usually the lower flange is made larger to obtain better use of both flanges in the working stage).

Now if, for some reason, the elastic method is preferred (over the plastic) then, in the writer's opinion,[14,15] the allowable moment should equal $M'_{L.el} = M_{L.y}/K = 387/2 = 193.5$ ton-meters > 152.

By this approach the required safety is provided against the appearance of the *load* producing the first yield. The allowable live load would then be represented by $M'_{L.el} - M_D = 193.5 - 100 = 93.5$ ton-meters as against $M_{L.el} - M_D = 152 - 100 = 52$ ton-meters.

The ultimate (critical) plastic carrying capacity of the section (in Fig. D, $0.85\, f_c' = 180$ kg per sq cm) equals $M_{L.cr} = 574.4$ ton-meters.

Assuming again $K = 2$, the allowable total moment by the *plastic* method would equal $M_{L.pl} = M_{L.cr}/K = 574.4/2 = 287.2$ ton-meters.

According to ACI-ASCE 333 recommendations the allowable total moment (see Fig. B) equals $M_{L,ACI} = 52 \times 1200/293 = 213$ ton-meters < 287. The loading history based on this recommendation is shown in Fig. C by dashed lines (parallel to the respective solid ones). The stresses produced by the different moments are summarized in Table A.

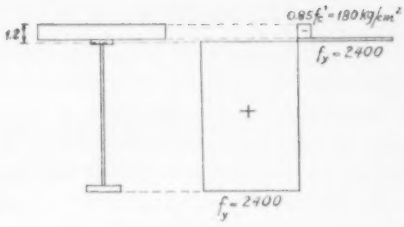

Fig. D — Ultimate resistance of section

While the example does not justify general conclusions, it still demonstrates that Section 402.2 produces a safe design (which may even be too safe) but does not convey a clear physical picture.

TABLE A — EXTREMAL STRESSES PRODUCED BY DIFFERENT MOMENTS (see Fig. C)

Moment, ton-meters	Steel stress Kg per sq cm		Concrete stress, Kg per sq cm
	$+f_1$	$-f_2$	$-f_3$
$M_D = 100$ resisted by steel beam only	785	1100	—
$M_{L,el} = 152$ allowable total moment by conventional elastic method	1078	$1200 = f_s$	13.8
$M'_{L,el} = 193.5$ allowable total moment by modified elastic method proposed by writer	1311 ($1.09f_s$)	1280	≈25
$M_{L,ACI} = 213$ allowable total moment by ACI-ASCE 333 Recommendation 402.2	1421 ($1.18f_s$)	1317	≈30
$M_{L,pl} = 287.2$ allowable total moment by plastic design method	1840 ($1.53\ f_s$)	1460	≈50
$M_{L,y} = 387$ total moment producing first yield	2400 (f_y)	≈1650	≈76
$M_{L,cr} = 574.4$ ultimate plastic moment	2400 (f_y)	2400 (f_y)	180 ($0.85\ f_c'$)

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14. Fountain, R. S., and Viest, I. M., "Selection of the Cross Section for a Composite T-Beam," *Proceedings*, ASCE, July 1957 and Discussion, Jan. 1958.

15. Zaslavsky, A., "Safety Factor in Structures in the Elastic Range without Stress-Load Proportionality," *Bulletin*, Research Council of Israel, V. 6C, No. 2, 1958.

CLOSURE By I. M. VIEST and P. P. PAGE, JR.*

In preparing the tentative recommendations, the committee set for itself two rules: (1) the recommendations should be written in terms familiar to most designers and (2) the design methods should be simple. The method recommended for the design of steel-concrete beams, using working stresses and neglecting the effects of shoring and time, satisfies both requirements. It also provides a design that is both safe and more economical than the conventional working stress procedure. It is true, however, that this end result was achieved at the sacrifice of consistency with the actual behavior at working loads. The committee was fully aware of this fact but considered that the advantages of familiar terms and simplicity outweighed this disadvantage.

Mr. Zaslavsky proposed two alternate procedures: design based on the yield moment and ultimate strength (plastic) design.

Yield moment design, though logical, is neither simple nor common in current practice.

Ultimate strength design is simple and logical. The committee considered this method in connection with the tentative recommendations but decided against its inclusion primarily for two reasons. At the time the recommendations were written, the plastic design of steel was just being published and could not be considered a generally-accepted procedure; as a matter of fact, it is not common in design circles as yet. Furthermore, factual information needed for the development of an ultimate strength design procedure for composite steel-concrete beams was not available, in particular, data on the design of the shear connection on ultimate strength basis were lacking. Several investigations were established, in part in response to the action of this committee, to provide the information needed for an ultimate strength design. The committee expects to prepare such a design method when the necessary data become available.

Mr. Firnkas' comments on various specific items in the report represent a welcome addition to the report. Particularly his emphasis on deflection considerations calls the attention to an important problem. It should be noted, however, that in addition to Section 107 deflection considerations are also included in the report of ACI-ASCE Committee 323[1] referred to in Sections 104.2 and 301 of the tentative recommendations for composite beams.

Section 202.3 on deformational stresses does not apply to prestressed concrete, and the recommendations for the effective flange width given in Section 105 conform to the provisions of Section 705 in the Building Code (ACI 318-56).

The committee wishes to thank the discussers for their contributions. The discussions should be helpful in the work on the final report.

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*Former committee chairman and chairman of task committee for preparing the recommendations, respectively.

Discussion of a paper by Walter H. Price, L. P. Witte, and L. C. Porter:

Concrete and Concrete Materials for Glen Canyon Dam*

By V. M. MALHOTRA, M. R. VINAYAKA, and AUTHORS

By V. M. MALHOTRA†

The authors are to be congratulated for the extremely interesting paper and for introducing the use of the heavy media separation process to remove soft undesirable particles from concrete aggregates.

While mentioning the use of tunnel concrete the authors made a reference to the use of a lignin type water-reducing agent and a corresponding saving in cement. From the results given by the authors it appears that they were obtaining an average compressive strength of 1310 psi more than the designated average strength of 3450 psi. It would be interesting to know if such a wide margin between design strength and field strength is normal for United States Bureau of Reclamation work and if not what considerations prevented the authors from increasing the W/C ratio to reduce this margin and thus reduce the consumption of cement. It is assumed of course, that the designed strength was based on 20 percent rejects and a coefficient variation of 15 percent.

The writer is interested in finding out if the six 4-cu yd mixers are being subjected to mixer performance tests and if so what is the frequency of such tests. Furthermore, since the discharge time of these mixers is probably of the order of a few seconds, thus eliminating the possibility of obtaining samples during discharge of the mixers, what special methods are being adopted to collect samples from the front, middle, and end of the mixers?

By M. R. VINAYAKA‡

The authors have done an excellent job in bringing out such a comprehensive article on the materials for one of the largest dams in the world. I shall deem it a favor if the authors clarify some of the following points:

*ACI JOURNAL, V. 32, No. 6, Dec. 1960 (*Proceedings* V. 57), p. 629. Disc. 57-30 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 12, June 1961 (*Proceedings* V. 57).

†Member American Concrete Institute, Geocon Limited, Montreal, Que., Canada.

‡Member American Concrete Institute, Research Officer, Research Division, Koyna Project, Koyna, India.

It has been mentioned that the design mix is expected to yield a laboratory strength of 3450 psi. This strength is to be obtained on 18 x 36-in. cylinders undergoing mass curing. What has been the field practices for casting and testing at 28 and 90 days? What has been the limitation imposed to control the quality as reflected by tests on 6 x 12-in. cylinders undergoing the standard curing which will ensure that 80 percent tests will yield strength higher than 3000 psi at 180 days on 18 x 36-in cylinders.

The coefficient of variation has been taken as 15 percent at 180 days. I feel that this is a high figure when keeping in mind the recommendations given in "Recommended Practice for Evaluation of Compression Test Results of Field Concrete (ACI 214-57)."* Normally it is expected that the coefficient of variation decreases with age in lean concretes. This leads to the view that a coefficient of variation higher than 15 percent is expected at earlier ages. It would be appreciated if the authors could indicate the coefficient of variation expected or being obtained at 28 and 90 days.

As regards permeability, it was mentioned that investigations were still in progress. It is expected that by this time the permeability tests must have been completed and that the authors would be in a position to state what the order of permeability has been for the interior concrete. The writer would like to know whether permeability tests were carried out in the Glen Canyon Dam interior concrete, including the largest size of aggregate, and without adopting the wet screening procedure. Have there been any criteria fixed by the United States Bureau of Reclamation for evaluating the quality of concrete on the basis of permeability? What is the permissible maximum permeability coefficient for the interior and exterior concrete for the laboratory specimens and the in situ tests on drill holes? For reflecting the deficiencies in construction joints, consolidation etc., are there any field-permeability test procedures detailed by the Bureau for dams?

Similarly for the creep studies, would it be in order to assume creep of concrete reflected by tests on small size cylinders prepared with wet-screened concrete to be the same as with the largest size included?

Clarification on the above points by such authorities as Messrs. Price, Witte, and Porter would undoubtedly be a valuable contribution for concrete technology and for proper assessment of results.

AUTHORS' CLOSURE

We are grateful to Messrs. Vinayaka and Malhotra for the questions raised because they afford us an opportunity to clarify portions of our paper which may have troubled others.

*ACI Committee 214, "Recommended Practice for Evaluation of Compression Test Results of Field Concrete (ACI 214-57)," ACI JOURNAL, V. 29, No. 1, July 1957 (*Proceedings* V. 54), pp. 1-19.

Mr. Vinayaka has questioned how the strength requirement of 3450-psi concrete is assured through field tests. Test cylinders, 6 x 12-in., are made from concrete wet-screened through a 1½ in. square-opening screen at the site and compressive strengths on moist-cured specimens are determined at 3, 7, 28, 90, and 180 days. From extensive correlation tests on concrete specimens containing the project materials, it was determined that an average 28-day strength of about 2800 psi must be developed by the job-made 6 x 12-in. cylinders so that development of 3450 psi at 180 days in 18 x 36 in. mass-cured companion cylinders might be expected. This information, of course, is being used by field personnel for their guidance in concrete control.

Mr. Malhotra's observation that a high factor of safety of approximately 1300 psi appeared to have been used for tunnel lining concrete is not strictly true. Here, for the sake of brevity, we apparently failed to present a completely clear picture in our paper. Actually, field control was achieved as follows: Design strength for the tunnel concrete containing 2½ in. maximum size aggregate was set at 3000 psi at 28 days. With a coefficient of variation equal to 15 percent, minimum average compressive strength for tunnel lining concrete would need to be 3450 psi. Control test cylinders cast on the job were 6 x 12 in. and were cast from concrete wet-screened to contain 1½ in. maximum size aggregate. Early on the job, an average strength of 4750 psi was obtained from 22 6 x 12 in. control cylinders cast from wet-screened concrete containing no water-reducing retarder. For months thereafter, and after adding a water-reducing retarder and lowering the cement content by 9.1 percent, the average strength obtained from 186 6 x 12-in. wet-screened cylinders was 4340 psi at 28 days. Here again, results of correlation tests indicated that compressive strengths obtained from 8 x 16-in. cylinders containing concrete of 2½ in. maximum size aggregate were only 85 percent of the strength obtained for companion concrete which had been wet-screened to 1½ in. maximum size aggregate and tested in 6 x 12-in. cylinders. The average strength of 4340 psi obtained from 6 x 12-in. cylinders when reduced by 15 percent equals 3690 psi. In the final analysis then, the strength of concrete lining when reduced to that indicated by 8 x 16-in. cylinders containing the full mix with 2½ in. maximum size aggregate was only 3690 psi, or 240 psi higher than the minimum average strength requirement of 3450 psi.

We agree with Mr. Vinayaka that a coefficient variation of 15 percent is high for a well-controlled job. However, it has been our experience that on a large job extending over several years, where large variations in temperatures occur and where there may be some variations in the materials during the period of construction, that an average coefficient of 15 percent should be used until sufficient data have been obtained to determine what control is actually being obtained on the

job. At Glen Canyon, because of difficulties we have experienced with the batching and weighing equipment, it was necessary to increase the cement content above that which was indicated as being necessary in laboratory tests. As improvements were made on the job, the cement has gradually been reduced and has now approached that which we feel will give us the desired strength at minimum cement content. Because of this gradual change in cement, we have not as yet established control data for this job. Generally, the coefficient of variation decreases with increased strength. However, it does not necessarily decrease with age. This point is discussed in "Precision Indices for Compression Tests of Companion Concrete Cylinders."*

We erroneously conveyed the impression that the permeability tests of Glen Canyon concrete were in progress. These tests have not yet gotten underway because it was necessary to move our permeability equipment. We expect these tests to begin soon so that we will have record information on the permeability of Glen Canyon concrete. We expect the permeability rate of the Glen Canyon concrete to be of negligible magnitude based on many tests made on similar concrete. Reference is made to Fig. 4 of the paper, "Development of High Quality Concrete of Low Cement Content Built by the Bureau of Reclamation."† This figure shows that the permeability even of lean concrete is small. No criteria have been fixed for evaluating the permeability of either interior or exterior mass concrete. Evaluation is principally by comparison with other concretes previously tested. The Bureau has not required in situ permeability tests on drill holes in field concrete and, therefore, has detailed no procedures to cover such tests. In general, our experience indicates that concrete made with well-graded aggregate and containing sufficient portland cement to furnish the required strength and which has been carefully placed and compacted, is, under laboratory conditions of test, sufficiently impervious to the flow of water. Any objectionable leakage that has occurred in our structures has been through cracks or construction joints. For this and other reasons, great pains are taken to obtain a good bond at construction joints and to eliminate cracking through control of the concrete construction.

With reference to creep of concrete, 6 in. diameter concrete specimens prepared with wet-screened concrete can be expected to develop greater creep strains than larger cylinders containing the full maximum size aggregate. This is reported in "Creep of Mass Concrete."‡

*Wing, S. P.; Price, Walter H.; and Douglass, Cleumeunt T., "Precision Indices for Compression Tests of Companion Concrete Cylinders," *Proceedings*, ASTM, V. 44, 1944, pp. 839-879.

†Price, Walter H., and Cordon, William, "Development of High Quality Concrete of Low Cement Content Built by the Bureau of Reclamation," *Proceedings*, Fifth Congress on Large Dams of the International Commission on Large Dams.

‡"Creep of Mass Concrete," *Miscellaneous Paper* No. 6-132, Report No. 3, Jan. 1958, United States Army Engineers, Waterways Experiment Station, Jan. 1958, Vicksburg, Miss.

We are now fabricating creep equipment to load 18-in. diameter cylinders, and when fabrication is completed, creep tests on Glen Canyon Dam concrete will be performed on both 18 x 36-in. cylinders, with full mass mix, and on companion 6 x 16-in. cylinders made with concrete wet-screened to contain 1½ in. maximum size aggregate. Information from these tests will be used in the study of the behavior of the dam as indicated by strain meters installed in the structure.

The mixer efficiency tests of the six 4-cu yd mixers mentioned by Mr. Malhotra are being performed. Mixer efficiency tests are scheduled to be performed on each of the six mixers once each week. As might be expected, sampling for mixer efficiency tests has required utmost cooperation between Bureau and contractor personnel. Because of the manner in which the mixers are supported and arranged it was difficult to obtain a sample of concrete as the mixers discharged. At the start of concrete mixing concrete samples were obtained from the collecting hopper under the mixer by discharging a small quantity of concrete at a time from the mixer into the hopper. Results obtained by this method were not satisfactory because of disturbance of the sample as it passed through the hopper. Samples were then obtained by a man actually entering the mixer and shoveling out the sample desired. Recently, alterations have been made and mechanical samplers have been installed under each mixer so that samples may be collected while the individual mixer is discharged a bit at a time.

Discussion of a paper by James G. MacGregor, Mete A. Sozen, and Chester P. Siess:

Effect of Draped Reinforcement on Behavior of Prestressed Concrete Beams*

By P. W. ABELES, FRITZ LEONHARDT, and AUTHORS

By P. W. ABELES†

The paper contains important research results which might appear most surprising and contrary to the accepted views by those who have so far based the design primarily on working load conditions. When, however, the state of cracking is considered, it seems understandable that by draping the tendons the depth available for diagonal cracking is reduced after flexural cracks have developed, and this has been clearly shown by the tests of the authors (Profiles A to C). It would, however, be expected that by leaving some of the tendons horizontal or by adding a separate horizontal reinforcement, the state of flexural cracking would be limited and consequently the resistance at diagonal cracking increased (Profiles W to Z). Surprisingly, the authors' tests have shown that this is not the case, and it would be important to investigate further whether that was caused by special conditions inherent in the test specimens in question (e.g., low concrete strength) or whether this can be considered as of a general nature. It would be interesting to hear the views of the authors in this matter. The writer would like to ask the authors why they have varied the concrete strength of the individual tests to such a great extent.

Another point to be considered would be the effect of a parabolic cable counteracting a uniformly distributed load. With such an arrangement the bending moment due to load is directly counteracted, and the effect of combined shear and bending does not come into full play as with the tests carried out. The writer would be interested in learning whether the authors have carried out any test with parabolic cables or with draped reinforcement and uniformly distributed load.

Although it would be more difficult to carry out such a test to failure, it would be important to do so to ascertain whether the conclusions drawn by the authors apply also to uniformly distributed loading or not, (with the latter case more likely).

The paper must be highly commended because of its clear description of the behavior of prestressed beams under flexural and shear

*ACI JOURNAL, V. 32, No. 6, Dec. 1960 (*Proceedings* V. 57), p. 649. Disc. 57-31 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 12, June 1961 (*Proceedings* V. 57).

†Member American Concrete Institute, Structural Consultant, London, England.

and the lucid distinction of the various types of cracking and the mechanism of failure following the initiating cracks. There is no doubt that the initiating crack at diagonal cracking in the web, which the authors call "web-shear crack," is caused when the principal stress exceeds the tensile strength, as the authors state. However, the writer cannot understand why the authors relate the actual tensile strength of the concrete to the nominal figure of a modulus of rupture, the magnitude of which depends entirely on the dimension of the specimen and the loading arrangement. The tensile strength has no definite relationship to the modulus of rupture or the compressive strength and this varies for different mixtures considerably for the same compressive strength; it does not, therefore, seem to be plausible to relate the tensile strength to the cylinder strength or to the modulus of rupture—the latter, itself, is only a nominal value, as already stated. Moreover, when the tensile strength is related to the root of the cylinder strength as $k\sqrt{f_c'}$ as it is sometimes done, this gives a quite wrong impression since the dimension is in lb per sq in. and the coefficient k would have to be adjusted to give the right dimension. It would, in the writer's view, be much simpler to introduce the tensile strength which is a quite definite value. By introducing the splitting test some idea of the actual tensile strength can be obtained and the writer would suggest the introduction of the tensile strength rather than its substitution by another strength for which no definite satisfactory relationship is known.

Further, the writer would like to refer to Eq. (5) and ask the authors whether they are of the opinion that it suffices to design the required stirrups for $V_u - V_c$, i.e., the ultimate shear capacity less the shear at inclined cracking. In the writer's view the results of various recent research have indicated that this is the case, but the writer wonders whether the same results would be obtained if there were some repetition of loading and he thought the required stirrup reinforcement should be based on the entire ultimate shear V_u so as to be at the safe side.

Finally, the writer would like to congratulate the authors for this work which throws some light on this rather, as yet, unsolved problem of shear resistance of prestressed concrete beams.

By FRITZ LEONHARDT*

This report on 22 test beams gives a great lot of valuable information on the behavior of pretensioned prestressed concrete beams. The authors must be congratulated for the intelligent and diligent way of the test and measuring procedures. The evaluation of the test results can, however, be done in different ways, leading to different results.

Most of the failures have not much to do with the profile of the prestressing wire, because the failure occurred immediately after the in-

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clined crack showed up. A great part of these failures were due to splitting of the concrete along the wires or due to yielding of the bond anchorage of smooth wires with their low bond strength. These modes of failure are quite natural for beams without web reinforcement.

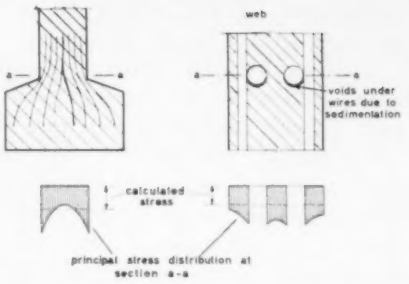

Fig. A—Stress distribution in web

In the writer's opinion, transverse (vertical and horizontal or helical) reinforcement should never be omitted at the zone of bond anchorage and the necessary amount should be computed depending on the location of the anchorage zone in the section of the beam. The draping of the wires can be used to distribute the wire ends over the end section, so that the amount of anchor reinforcement will be small. A concentrated anchorage of straight wires at the bottom of the section requires more vertical reinforcement and leads to unfavorable stresses. Of course, it can be more economical to place this additional reinforcement than to drape the wires, but this reinforcement is a necessity. The calculation of this anchor reinforcement under the favorable effect of the bearing reaction can be done along the lines reported in a PhD thesis made at my chair in 1960 by M. Sargious.

The initial inclined crack is caused by the diagonal principal tensile stress and the load V_c causing it, is taking an important place in the report. It increases with draping. The test results of V_c compare fairly well with the theoretical ones. However, under practical conditions, there are many other reasons for tensile stresses in the web than loads, i.e., shrinkage and temperature stresses and differences in creep. Therefore, this V_c is not a value which can be relied on. Further, the stress flow between flange and web, will always give an unequal stress distribution at the beginning of the web with high local stresses (Fig. A).

A similar effect have draped wires in the web, especially if high W/C ratios (up to 0.9 have been used!) cause voids below the wires. The horizontal cracks between flange and web even in the flexural zone of some beams prove that beams without a reasonable web reinforcement should not be used to judge the influence of draping. That beams without web reinforcement behave a little better with straight wires than with draped ones, is to be explained by the fact that, without stirrups, the only way to carry a load after the inclined cracking has begun, is by the tied arch action. This arch action is better with a straight tie than with a draped one.

The paper reports that draping decreases the inclined cracking load for flexure-shear cracks. This is true for those beams without straight wires, where the bottom edge was practically without longitudinal re-

inforcement. Edges, which get tension, should, however, always be reinforced with a small amount of wires or bars.

If we exclude all beams with such design deficiencies, then the beams with draped wires show a good behavior and fail in flexure even with a small amount of stirrups. The advantage of draping is, however, not important for the load capacity of small slender beams, it is to be seen in better anchorage conditions. Draped tendons become more advantageous for larger beams, especially if high loads in a fixed position have to be carried or if the eccentric introduction of the concentrated prestressing force would cause difficulties.

AUTHORS' CLOSURE

The authors perceive in both Dr. Abeles' and Professor Leonhardt's remarks a concern about the atypical properties of the test beams: low concrete strength, lack of adequate reinforcement, linear drape profiles with all the reinforcement draped. The test beams were designed to bring out measurably the effects of draping the longitudinal reinforcement, not to simulate ordinary prestressed concrete beams. A well-proportioned beam should not fail in shear. To determine how a beam can be designed to fail in flexure, it is necessary, though not sufficient, to observe the behavior of beams failing in shear with and without web reinforcement. One of the most important deterrents for the development of a reasonable design procedure for stirrups in ordinary reinforced concrete beams in the United States was that too many of the test specimens had been typical and had failed in flexure giving no indication of the optimum amount of web reinforcement required. The primary reason for the choice of low concrete strengths and total drapes was to exaggerate the effects of inclined tensile stresses so that quantitative trends could be observed reliably. These trends were checked against and confirmed by tests on beams with high concrete strengths, partially draped reinforcement, and web reinforcement.

Dr. Abeles' question about the feasibility of providing web reinforcement to "carry" the quantity $V_u - V_c$ rather than V_u (where V_u and V_c are values of the shear corresponding to the ultimate and inclined cracking loads, respectively) is one of both theoretical and practical interest. At the turn of the century, Talbot observed that not all the shear need be assigned to the web reinforcement. None of the test results known to the writers has refuted this observation. A term V_c has been incorporated in most codes of practice in the United States, though with somewhat fluctuating values and weathervane justifications. In 1957, Hernandez' work on prestressed concrete beams[2] showed a close relation between V_c, the inclined cracking shear, and the shear "carried by the concrete." Further tests at the University of Illinois have confirmed this trend. Nevertheless, a completely satisfactory physical in-

terpretation of the "shear carried by the concrete" is still lacking. That only a portion of the shear should be assigned to the web reinforcement is certain. That this shear is related quantitatively to the shear at inclined cracking appears quite likely on the basis of experimental evidence.

Tests were carried out under simulated moving loads to check the applicability of this approach to the design of web reinforcement in highway bridge girders.[3] In these tests a single concentrated load was moved along the beam. The load was increased in small increments after each "pass" until failure was reached. These tests also confirmed the observation that the shear corresponding to inclined tension cracking could be assigned to the concrete even if the load causing the inclined crack is removed and then reapplied. However, the results of these tests should not be projected to cover the case of repeated (fatigue) loads. Of course, the web reinforcement may be designed to carry only the shear $V_u - V_c$ if the beam is not cracked under repeated loads. However, if there is a likelihood of inclined cracks under such loads, it would be prudent to provide web reinforcement to carry all the shear and at a low working stress unless evidence exists to justify reduction in the amount of web reinforcement.

For a uniformly-loaded beam with parabolic cables, it is quite likely that the critical crack will be a web-shear crack. In that case, the drape may have a beneficial effect on the "shear strength" of the beam. However, this is an effect of the loading condition and not of the shape of the drape.

The writers concur with Dr. Abeles in his desire to see the "cylinder splitting test" become a standard test for concrete. The test is easy to perform, gives consistent results, and requires only a small amount of concrete (6 x 6-in. cylinders give quite good results[4]). Until this test becomes a standard, however, the test results must be related to the cylinder or cube strength if they are to have any meaning at all for the practicing engineer.

The authors do not understand Dr. Leonhardt's statement, "most of the failures have not much to do with the profile of the prestressing wire, because the failure occurred immediately after the inclined crack showed up." The profile of the reinforcement affected the inclined cracking load and, therefore, the failure load. The nature of the failure was also affected. A beam without web reinforcement is often able to sustain further load after inclined cracking through arch action, although this increase in load is erratic and unreliable. As Dr. Leonhardt remarked, the straight tie (straight reinforcement) makes for a more efficient arch than a draped one. Therefore, failure is more likely to occur immediately after the inclined crack in beams with draped reinforcement than in beams with straight reinforcement. Furthermore, the connection between the "tie" and the "arch rib" is relatively weaker

in beams with draped reinforcement if it is effected through bond, since bond gets poorer with increase in depth of concrete under the reinforcement (References 5, 6, and 7).

A decrease in the inclined cracking load was observed for beams with partially draped reinforcement as well as for beams with all the reinforcement draped. The data plotted in Fig. 10 show the close correlation between the observed flexure-shear cracking load and the load corresponding to the computed flexural cracking load at a distance x from the load point ($x = a/6 + h/4$, where a = shear span, h = beam height). The computed quantity V_f is a function of the eccentricity of the *total* prestressing force at the distance x from the load point. Thus, the observed reduction in the inclined cracking load was essentially the same for comparable beams having the same eccentricity of the total prestressing force at Point x, whether all or part of the wires were draped.

Dr. Leonhardt's thoughts on the differences between beams in the laboratory and in the field deserve special emphasis. Unfortunately, there have been too many recent examples of applying "numbers" obtained in the laboratory from isolated members, under controlled conditions of manufacture and loading, directly in design methods. However, the writers are not skeptical enough to conclude that because one cannot calculate the stresses precisely one should not try to calculate them.

The significant conclusion of the tests was that draping does not always increase the shear strength. The tests should not be interpreted as indicating that beams with draped reinforcement are not to be used; if anchorage conditions or deflection limitations call for a drape, the beam *can* be designed with draped reinforcement and a suitable amount of web reinforcement to insure a flexural failure. However, the designer should consider the conditions rather carefully before draping the reinforcement solely to increase the shear strength.

In conclusion, the writers wish to thank Dr. Abeles and Professor Leonhardt who have enriched the paper by their pertinent and penetrating comments.

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Discussion of a paper by O. P. Jain:

Ultimate Strength of Reinforced Concrete Arches*

By GIULIANO AUGUSTI, BINOY KUMAR CHATTERJEE, W. T. MARSHALL, T. K. PANDIT, and AUTHOR

By GIULIANO AUGUSTI†

The limit analysis and design of reinforced concrete arches is an important problem, not yet dealt with sufficiently in current literature. Dr. Jain's paper is therefore welcome, since it supplies a good deal of useful knowledge on the matter, and gives interesting experimental evidence. I think that a few critical remarks may be of some interest, without diminishing the value of Dr. Jain's work.

First, in the writer's opinion, the limit analysis of reinforced concrete structures (as well as steel structures) can be pursued most successfully using simple plastic theory, i.e., by identifying the point of collapse with the formation of a plastic hinge mechanism. Several reinforced concrete structural models, tested at the University of Naples, collapsed all by formation of such a mechanism; the same behavior has been observed by other investigators. In no experiments ever—as far as the writer knows—the decrease in the plastic moment in the hinges, due to exaggerated deformations has been sensibly bigger than the uncertainties one must expect when dealing with structural concrete; although it must be recognized that this could occur in some limit cases. As a matter of fact, Dr Jain himself reports that in his tests "there was no visible crushing of concrete even at the ultimate load" and collapse occurred by formation of a mechanism.

Therefore I don't think it is worthwhile to follow the much more complicated "limit deformation" approach to the limit analysis of concrete structures, as Dr Jain and some other authors do.

In applying the plastic hinge theory to arches, it is of course necessary to take into account the influence of the axial load on the value of the full plastic moment and on the location of the neutral axis (the latter effect, however, is often inessential). This is easily done if a rectangular stress block of compression concrete is assumed, as first suggested by Whitney[9] as far back as in 1937, and since then proved to be the sim-

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plest, and at the same time accurate, assumption. Recently, such formulas for several shapes of sections and distributions of reinforcement have been summarized by Franciosi[10] and confirmed once more by experiments.[11,12]

When the plastic moment versus axial load curves for the sections of a given arch have been obtained, the collapse factor of whatsover distribution of loads can be found by a trial-and-error procedure,[10] which involves the alternate determinations of (1) tentative upper bounds of the load factor, and (2) values of axial load in the plastic hinges. In a recent paper,[13] the ultimate value of a concentrated force acting on a model fixed-end arch was evaluated in this way, and a satisfactory agreement between theoretical and experimental values was found.

Nevertheless, while the deformation approach is not essential for the determination of the ultimate load factor, at the same time it is the only approach which allows for the determination of the stress distribution in the elasto-plastic range before collapse. This may be perhaps the most fruitful application of procedures such as Dr Jain's.

However, it must be noticed that Dr Jain calculates the effects of plastic deformations neglecting the plastic elongation of the arch axis, and taking into account the plastic curvature only. This is accurate enough in his case of a concentrated load only, because in such a loading condition the effect of $\Delta\phi$ in the positive moment zone (near the point of application of the load) is much bigger than the opposite effect of $\Delta\phi$ in the negative moment zone. The usual severest condition of loading of an arch (say, of a bridge arch) is however a uniform load spreading over half span. In this case the maximum positive and negative moments are almost the same, if the shape of the arch axis is approximately parabolic as is usually the case; therefore the two opposite effects of $\Delta\phi$ almost eliminate each other and the effect of the elongation of the axis predominates.

In a recent analytical study, carried out at the University of Naples,[14] a two-hinged concrete arch was analyzed in the plastic range. The data of the arch is as follows:

Span: 60 m (197 ft)

Rise: 12 m (39 ft)

Section: Rectangle, 200 x 100 cm (78.8 x 39.4 in.)

Reinforcement: 15 bars, 30 mm (1.18 in.) diameter, along each horizontal edge

Dead load (assumed to be uniform): 7.00 metric tons per meter (4.70 kips per ft)

At a live load of 22.50 tons per m (15.15 kips per ft), spreading along half span, the horizontal thrust was found to be the sum of the following terms:

(a) Main elastic thrust (neglecting arch shortening):	+683.5 tons (1508 kips)
(b) Effect of elastic arch shortening:	− 3.25 tons (7.2 kips)
(c) Effect of $\Delta\phi$ in positive moment zone:	+13.16 tons (29.0 kips)
(d) Effect of $\Delta\phi$ in negative moment zone:	−13.10 tons (28.9 kips)
(e) Effect of plastic arch elongation:	+ 2.23 tons (4.9 kips)

As is seen from the above table, the effect of $\Delta\phi$ nearly vanishes (although each plastic zone spreads over about one-third of the arch length), while the effect of the plastic arch elongation is noticeably bigger (although still negligible in comparison with the total elastic thrust).

In this investigation, the determination of the elasto-plastic stress distribution was approached by a trial-and-error procedure based on the influence lines of the horizontal thrust for distortions. This procedure was a natural extension to arches of an analogous one outlined elsewhere for simply bent structures:[15,16] it was sometimes found to shorten the tedious direct calculation of the effects of inelastic deformations, and is especially suitable when several load distributions have to be analyzed for the same structure, because the preliminary work does not have to be repeated.

To apply this procedure to the mentioned example, the formulas of the moment-curvature and moment-elongation relationships for a rectangular concrete section, the value of the axial load being fixed, were first of all determinated. The assumptions, as usual, were that plane sections remain plane and that the stress-strain relations of concrete and reinforcement are as in Fig. 4 (but no account was taken of a limit to the deformation).

Curves were then drawn for the particular section of the arch and several values of the axial load in the probable range. It was thus possible to read on the graphs the values of plastic curvatures and elongations, as a difference between the total and the elastic (straight line) values.

On the other hand, the influence lines of the horizontal thrust for a unit variation of length and for a unit relative rotation were determined.

A typical cycle of the trial-and-error procedure was then as follows:

(a) Fix a reasonable value of the thrust and determine the corresponding stress distribution

(b) Read on graphs the plastic curvatures and elongations in sections at suitable intervals, and plot them along the span

(c) Multiply the values in (b) by the corresponding ordinates of the influence lines, and find the value of the thrust by numerical integration

(d) Add the thrust in (c), due to plastic deformation, to the elastic thrust, and compare with initial value of the cycle.

Three or four cycles were usually sufficient to attain a satisfactory accuracy.

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By BINOY KUMAR CHATTERJEE*

The paper is a valuable contribution in the field of ultimate-load theory as applied to reinforced concrete arches.

It was interesting to apply Dr. Jain's theory to the analysis of one large size arch designed by the writer[17] recently. This arch is two-hinged and has a catenary profile as shown in Fig. A. The span is 120 ft. 0 in. and the height is 74 ft 0 in. The arch acts as a stiffener to catenary shells forming the fertilizer silo in Rourkela, India, as shown in the construction photo Fig. B. Collapse load calculations were made on the basis of Dr. Jain's theory assuming the application of a concentrated load at quarter span. It was found that the collapse load was 47 percent in excess of that given by the elastic theory. In the particular example given by the author this increase is 86 percent. It is seen that for steeper arches like the catenary the excess is less than that for flat segmental arches or semielliptical arches. Hence a generalized statement that "ultimate strength . . . is 50 to 100 percent greater than that given by elastic theory" does not always hold.

The reasons for the reduction of increase of load bearing capacity in the plastic region can be attributed to the following.

In the case of steeper arches the scope for the redistribution of positive and negative moments, is smaller and therefore the load bearing capacity is not increased much over that calculated by elastic theory. Secondly as shown by Dr. Jain the excess depends on the eccentricity of thrust which again is a factor depending on the position of the load.

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For a particular type of loading, it may be that as the load exceeds the value calculated by elastic theory, there will be an inward movement of the ends of the arch. The net outward movement may be so small as not to allow considerable extra load bearing capacity over that calculated by elastic theory.

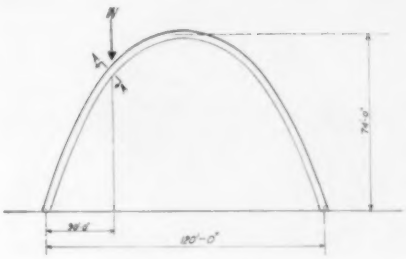

Fig. A(a)—Arch profile

The author has proposed to assume failure after the concrete has attained an ultimate strain of 0.3 percent, where there is no lateral restraint on concrete, and 0.6 percent or more in case of lateral restraint. These limiting values of strain according to Baker[18] are 0.1 and 1.0 percent, respectively. Subsequently Baker [19] limited the strain to 0.2 percent (without any particular reference to lateral restraint). According to the recent investigation of Poologasoundranayagam[20] the safe limiting value without lateral restraint can be assumed to be 0.35 percent. From the experience of the writer it seems that Baker's limitation is perhaps too conservative. The writer rather agrees with Dr. Jain and Dr. Poologasoundranayagam. The actual collapse load would be considerably higher than that calculated according to the limitations, laid down by Baker. It is however seen that Fig. 2 does not indicate the presence of stirrups. The writer therefore fails to understand how Dr. Jain has taken the value of e_u equals 0.6. Or, was it that there were stirrups in the structures actually tested.

The author has taken a value of E_c equals 3.84×10^6 psi. In a generalized manner, he has stated that the effective modulus of elasticity of concrete may be taken equal to about 1400 times the cylinder strength of concrete. It is quite interesting to compare the works of Sharma and Gupta[21] in this respect. According to the formula proposed by them the multiplying factor comes to 1430 instead of 1400 for a cylinder strength of 2760. This factor according to them varies with the varying

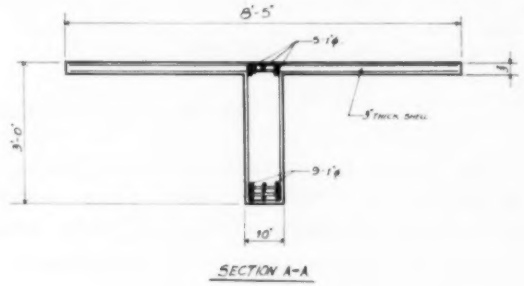

Fig. A(b)—Arch cross section

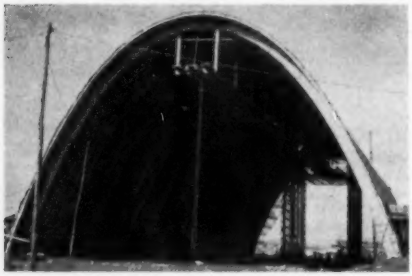

Fig. B—Arch under construction

strengths of concrete. It appears that according to Dr. Jain the factor remains constant at 1400 irrespective of the strength of concrete. The writer would like to know if Dr. Jain has conducted tests in arriving at this conclusion. The writer is interested in this, since in the design stage it is some times essential to forecast a correct value of the modulus of elasticity of concrete as related to the strength of concrete prescribed in the design of structures like prestressed concrete, reinforced concrete shells, etc.

Regarding the value of L_p, the author is asked to elaborate on how to establish the same as a function of span for different conditions of variation of moment.

The method of calculating the collapse load suggested by Dr. Jain is a useful one. But unfortunately this consists of a process of repeated trials. The writer wonders if it is possible to establish some mathematical expressions for direct calculations.

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By W. T. MARSHALL*

The author in his conclusions states that he has evolved a method of calculation which he claims gives "complete agreement" with experimental values. This is a bold claim to make in dealing with con-

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crete in the plastic range. Even if the claim is true, it appears to be based on the assumption that the maximum strain in the concrete is 0.6 percent. This is a much higher figure than is generally accepted in ultimate load design and makes the value of the theory debatable.

The main objection to the theory, however, is the amount of trial and error work involved in the calculations. First, one assumes a relationship between H and W and from this assumed ratio for H/W is calculated by equating the stresses on the section. Then using Eq. (3) to (7) the angular rotation is calculated leading to the $\Sigma\Delta\phi yds$ term and another value of H is now calculated. It is assumed that if this is less than that obtained from the initial assumption, then the initial value is taken as correct. In the example given there is only a 6 percent difference between the two values but what further calculations would the author have carried out if there had been say 20 percent difference between the two values of H?

The calculation of the $\Sigma\Delta\phi yds$ term is, as the author states, the tedious part of the calculations. In the paper the author makes an assumption for ϕ as 2×10^{-3} radians which happens to be correct, but how many other assumptions had he used before getting one which enabled the stress-strain relationship for the steel to be satisfied? This operation might be described as satisfying the compatability of strains. It is noticed that in this case the value for N is 0.244 while the value obtained for N when satisfying the compatability of stresses is 0.21. There is, the writer feels, too big a difference between these values for the author to claim that he has developed a "complete" theory. Incidentally, since a value has to be assumed for either ϕ or N in solving Eq. (3) to (7), why did the author not use the value of N already obtained from the stress compatability equations and solve for ϕ instead of making what must be an almost complete "shot in the dark" assumption for ϕ and then determine N?

The arch rib is essentially an eccentrically loaded column and much research has already been carried out for such members, the final results of which are summarized in the column formulas given in the ACI Building Code, Section A609. There seems little point in the author bringing forward another theory for dealing with such members.

The author rightly states that the failure of an arch rib will take place when four plastic hinges have been developed, provided of course that the hinge rotation at the third hinge is not excessive so that the fourth hinge can develop without complete failure of the member at the third hinge position.

In the case of the arch with a point load at quarter point, failure occurs when the moment under the load at O is equal and opposite to the moment at a point corresponding to F. Under these conditions sim-

TABLE A—COMPARISON OF ECCENTRICALLY LOADED COLUMN THEORY

Arch No.	Load Position	Reinforcement	Cylinder strength, psi	Test load, lb	Theoretical load, lb	Test / Theory
1	Quarter point	2-1/4" e.f.	2610	2644	2380	1.114
2		2-3/8" e.f.	2250	4280	4300	0.997
3		2-1/4" e.f.	4550	2882	2320	1.242
4		2-3/8" e.f.	4500	4440	4800	0.925
5		2-1/4" t.f.	2280	2520	2180	1.156
Table 4		2-1/4" e.f.	2760	2880	2300	1.38
7	Crown	2-1/4" e.f.	2310	4745	4840	1.020
8	Crown	2-3/8" e.f.	2445	7880	9500	0.830
					Average	1.083

Note: e.f. denotes reinforcement on each face; t.f. denotes reinforcement on tension face only.

ple statics can be used to determine H in terms of W, the relationship being given in the paper as $0.805W$.

There are then two sections of the arch, O and F, each subjected to the same bending moment and with almost identical direct thrusts (the slight difference in direct thrusts is due to the fact that the slope of the rib at F is slightly different from that at O).

Since H is known, both the bending moment and thrust can be given in terms of W and the eccentricity of the load, e, calculated. The thrust at the section corresponds to the load P_u given in Eq. (A9), (A10), and (A11) of Section A609. When e has been calculated all the terms on the right hand side of the equation are known and P_u can be determined. The relationship between P_u and W is given by: $P_u = W \sin \theta + 0.805W \cos \theta$ (where θ = slope of arch rib at the quarter point). Hence when P_u has been determined W can easily be found.

This method is quite easy to carry out and requires none of the trial and error approaches used in the theory given in the paper.

In his PhD thesis (Reference 2) the author gives the results of tests on seven 4 x 4 in. arch ribs, five of which were loaded at the quarter point and two at the crown. These arches together with the result given in Table 4 were analyzed by the simple eccentrically loaded column method given above and the results are given in Table A.

This relatively simple method of analysis gives an average result showing good agreement between theory and test. It is in error in general on the safe side and only gives a large underestimate in the case of Arch 8 where the theoretical load is high. In this case it is quite likely that the hinge rotation at the center became excessive before the remaining plastic hinges could develop and in addition the arch-shortening effect which has been neglected would be more appreciable when the load is high.

The writer considers that this simple eccentrically loaded column approach constitutes a reasonably reliable approximate method of dealing with the two pinned arch.

By T. K. PANDIT*

The paper by Dr. Jain is interesting and quite encouraging to me as I am preparing a thesis on the "plastic design of indeterminate reinforced concrete portal frames" at the University of Sheffield.

In establishing the relationship between the only redundant reactive force, i.e., the horizontal thrust H, and the externally applied load P on a two-hinged arch, it is assumed in the elastic theory that the horizontal span length between the two hinges must not change. This assumption, though theoretically ideal, seems to be inconsistent with the author's experimental method of measuring the horizontal thrust. I wonder whether there were any span-controlling devices in his apparatus to satisfy the above assumption for each successive loading. The errors, involved in measuring the horizontal thrust in an apparatus where there is no span-controlling device, might be small but not at all insignificant and can lead to accumulative errors in the ultimate load condition and especially the corresponding experimentally measured horizontal thrust. I fully appreciated these defects while designing my apparatus for measuring all the indeterminate forces in a fixed-ended single bay reinforced concrete portal frame under single vertical load and devised remedial techniques accordingly to measure the horizontal thrust.

The author should be congratulated for his concept of the possible differentiation of ultimate strains of concrete between laterally restrained and unrestrained conditions. I feel happy, however, about the value $e_c = 0.3$ percent in the case of laterally unrestrained concrete but beg to differ with the value $e_c = 0.6$ percent which seemed to be a most unconversative generalization in the case of laterally restrained ordinary reinforced concrete, irrespective of its strength and grade, without any attempt made to increase effective straining capacity of concrete in the compressive zone. I made repeated tests on many restrained cases of concrete and found the ultimate strain values not more than 0.45 percent where the cylinder strength of the concrete f_c' was found to be 6500 lb per sq in.

I should be pleased if the author could suggest any possible direct experimental technique to measure the "spread-up-plastic-length l_p" so as to justify its value obtained from the so-called established formulas with measured indirect experimental readings.

AUTHOR'S CLOSURE

The author is greatly indebted to all those who have taken part in the discussion on his paper and have produced valuable comments.

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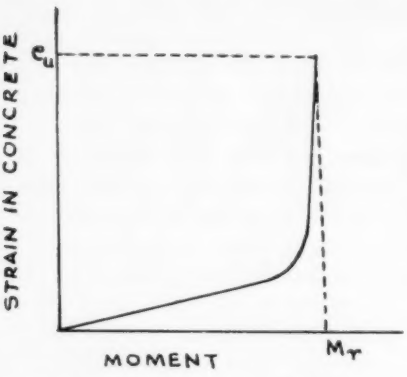

Fig. C—Moment-strain relationships

It is hoped that these comments will contribute much to a better understanding of the problem.

The first point raised by Professor Marshall is that an ultimate concrete strain of 0.6 percent used in the paper is much higher than generally accepted and makes the value of the proposed theory debatable. In this connection, attention is invited to the References 18, 22, 23, and 24, where strains as high as 1 percent or more have been reported for confined concrete and even unconfined concrete was believed to show strains as high as 0.5 to 0.6 percent. These strains were measured on cylinders and were uniform through their length. In the case of bending, the strains vary along the length of the member with bending moments. As the moment approaches the ultimate moment of resistance M_r of the section, the strains in concrete increase sharply as shown in Fig. C.

In the case of a frame or arch, where the moments decrease on either side of the section of maximum moment, the concrete strain in the immediate vicinity of the section tending to fail remains low, while at the section failing, it is manifold of this strain. Hence, theoretically the ultimate concrete strain occurs at one section only, i.e., in an infinitesimal length of the member. Obviously, the length of fiber subjected to the same ultimate strain is going to influence the value of this strain. If it is assumed that crushing of concrete occurs due to buckling of its' fibers, then a shorter length will be able to take-up more strain before failure. That appears to be the reason why laterally supported concrete takes more strain. Hence concrete in bending can withstand more strain than in a simple compression test of cylinders. As Whitney has stated, the brittle nature of concrete in a compression test is partly due to the sudden release of strain energy of the testing machine at failure of specimen. Thus tests on cylinders cannot give a true idea of ultimate strain of concrete and certainly not of the strains that may exist in beams. The maximum strain in beams is also not possible to measure by strain gages, which will only give an average strain in the gage length, while the maximum strain can be several times the average as per figure.

The general belief of 0.2 percent strain at failure of concrete came into being due to Baker.[6] But, Baker deals with average strains and not the maximum strain just over a crack. Hence in the author's opinion,

the recommended values of ultimate strain of 0.3 percent for laterally unsupported and 0.6 percent for laterally supported cases are on the conservative side.

The lateral support is not only provided by a close pitched spiral, but also by transverse loading. In the case of beams, the contact pressure between the load and the top surface of beam stops any spalling of concrete and provides the necessary lateral support in the loaded zone. This effect was clearly noticed in the tests on these arches. The concrete under the load (point of maximum moment) did not crush, whereas at the point of maximum negative moment, crushing of concrete was visible even though the moment here was slightly lower.

Professor Marshall refers to the two values of N calculated in the paper as not satisfying the conditions of compatibility of stresses and strains simultaneously, and thus he doubts the theory being "complete." Unfortunately, due to oversight, he appears to have mixed up the two values of N calculated for the two sections of maximum positive and negative moments. In fact, his doubt is unfounded.

Professor Marshall also objects to the labor involved in the trial and error process and suggests an easy approach based on the column formulas given in the ACI Building Code, Section A609. In fact, the author has already mentioned the tediousness of the method and has suggested a possible practical approach. But Professor Marshall has not appreciated the need of the trial and error process. It is not meant to calculate the thrust which a section can bear at a given eccentricity (this is all that Section A609 can do), but the main purpose is to calculate $\Sigma\Delta\phi yds$, so as to make sure that in the resulting moment redistribution due to plasticity of materials, the concrete is not put to strains more than it can bear. Professor Marshall's suggested approach ignores this aspect and assumes concrete to be as ductile as steel. In fact, with this assumption, the very purpose of the paper is defeated.

Mr. Pandit's doubts about the ultimate concrete strain being as high as 0.6 percent have already been commented on. Regarding the control of span, the two ends of the arch were joined by a tie rod and the only increase in span could be equal to the stretch of tie. This effect has already been taken into account in Eq. (1) and (2).

Regarding Dr. Augusti's comments, it will be a happy situation if we could apply the simple plastic theory to the analysis of reinforced concrete redundant frames. In that case, all trial and error will be eliminated. But this assumes that plastic hinges can undergo the necessary rotation without breakdown of concrete or loss of moment of resistance. This may be true to some extent in structures with one or two degrees of redundancy and may be unsafe for higher degrees. Hence the deformation approach will be necessary until it is shown either experimentally or theoretically that full rotation of hinges can

take place. In fact, the simple plastic theory provides a valuable tool to choose the data for the first trial. The actual bending moments will lie between the two limits given by elastic theory and simple plastic theory. After some experience in this respect, rules can be framed giving the extent up to which moments can be modified to bring them nearer to those given by plastic theory for various degrees of redundancy.

The author agrees with Mr Chatterjee that the excess in collapse load over that given by elastic theory will depend on the curve of the arch axis, rise-span ratio, pattern of loading, and variation of section of the rib.

In the actual model tested, stirrups were provided as is done in a normal design. Fig. 2, however, does not show these stirrups. But as explained earlier, the load itself provided the lateral support to the concrete at the place needed and thus concrete could take up strains as high as 0.6 percent.

The author feels that the value of modulus of elasticity will not always be 1400 times the cylinder strength, but will vary with the concrete strength and the stress up to which it is subjected. No special tests were conducted and guidance in this respect for purposes of design can be had from the various codes of practices.

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Disc. 57-35
(Including 57-11, 57-15, 57-22, 57-25)

Discussion of a five-part series by J. Taub and A. M. Neville:

Resistance to Shear of Reinforced Concrete Beams*

By P. W. ABELES, CHARLES ERDEI, PHIL M. FERGUSON, K. HAJNAL-KONYI, A. HELFGOT, C. J. POSEY, E. M. RENSAA, R. TAYLOR, W. WELEFF, and AUTHORS

By P. W. ABELES†

This series of papers shows again how controversial the position is with regard to the shear-resistance of reinforced concrete. So many points are raised, that it would be impractical to deal with all of them and in the following only a few points are discussed on general lines. With some of these the writer is in full conformity, while with others he disagrees.

It is incorrect, as the elastic theory states, that the shearing stress is influential with regard to diagonal cracking. The writer fully agrees with the authors that the design should be based on the principal tensile stresses. It appears, therefore, to be quite reasonable for the authors to suggest a calculation according to which the shear reinforcement should be evaluated to take up the ultimate principal tensile stresses which would occur if the section were homogeneous.

The writer is also in complete agreement with the authors' view that it is essential to have the stirrups closely spaced and to prefer a greater number of small-sized bars rather than fewer large-sized bars, so as to obtain a better distribution. This principle has been successfully introduced to reduce the extent of flexural cracking and should, therefore, be similarly applied to the placing of shear reinforcement.

The authors are of the opinion that with vertical stirrups the shearing resistance can be increased but the maximum that can be obtained with diagonal reinforcement cannot be achieved. The writer does not believe that this is correct. He agrees with the authors that it is possible to obtain the maximum shear resistance with a smaller amount of diagonal reinforcement and it is, therefore, more economical to provide such reinforcement. That does not mean that by the provision of

*ACI JOURNAL, V. 32: No. 2-6, Aug.-Dec. 1960 (*Proceedings* V. 57), p. 193, 315, 443, 517, 715. Disc. 57-35 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 12, June 1961 (*Proceedings* V. 57).

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sufficient adequately-spaced vertical stirrups, satisfactory shear-resistance cannot be achieved.

Another point which has puzzled the writer relates to bond failure. The authors state that bond failure is *always* of secondary nature. The writer may conclude from some experience which he has had with prestressed concrete that sufficient bond appears to be essential to ensure satisfactory shear and flexural resistance. If sufficient bond resistance is available he has experienced that bond failure occurred only as secondary failure, as the authors have described, i.e., after failure had primarily taken place due to flexure or diagonal tension. On the other hand, whenever the bond was insufficient the steel slipped before flexural or shear failure took place and in this case bond failure was the primary cause even if such slipping occurred shortly before the anticipated failure.

To clarify important problems the authors have discussed a great number of useful points some of which are, of course, well-known to specialists, but are not always borne in mind by the designers. There are obviously also some controversial points in the papers which may give rise to eventual clarification in the discussion. Therefore, the efforts of the authors must be commended.

By CHARLES ERDEI*

Notation

τ_{cu} = shear strength of the compression zone

f_c = $k_o f_c'$, ultimate compressive strength of concrete where k_o is the shape factor for assumed rectangle distribution

f_{cp} = compressive concrete stress acting parallel to the axis of the beam

f_{cb} = bearing strength of the concrete

$v = \dfrac{f_c}{\tau_{cu}}$

$n = \dfrac{v}{2b\,\tau_{cu}}$

δ = original inclination of shear reinforcement, measured between the axis of the beam and the axis of the inclined shear reinforcement

w = inclination of the normal of the principal crack to the axis of the beam

α = inclination of the shear reinforcement after the cracks have opened

t = horizontal projection of the inclined cracks

A = area of main tensile reinforcement

A_B = area of web reinforcement

M = bending moment acting on the cross section

V = shear force acting on the cross section

T = force in main tension reinforcement considered horizontal in this discussion

B = force in web reinforcement

p_w = permissible stress in the web reinforcement

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Many thanks are due to the authors for their excellent survey and presentation of the shear problem up to date. The writer is particularly impressed by the clear conception the authors have about the problem as a whole.

The writer was employed until 1956 with the Hungarian Ministry for Building Affairs and there he did research on the shear resistance of reinforced concrete members. He was also a member of the structural committee of the Hungarian Standard Institute which prepared the new standard specifications based on ultimate load theory. The writer would like to make some observations about the interpretation of the research results and make some notes for practical design purposes.

The writer does not believe that the specimens quoted in Table 1 for large a/d ratios (e.g., Morrow's and Viest's Specimens B56B4, B84B4, B113B4R, B56B2, B70B2, B56B6) failed in shear but instead failed in bending as the following will prove. Take for instance the case of the rectangular beam loaded with a concentrated load at midspan similar to Morrow's and Viest's examples. The bending moment resistance of the beam according to Section A605, Eq. (A1) and (A2) of the ACI Building Code* will be $M_r = [bd^2 f_c' \ 0.4 \ (1 - 0.59 \times 0.4)] \ 0.9^2 = 0.9^2 \ d^2 f_c' b \ 0.306 = 0.246 \ f_c' bd^2$ we had to use (14.5/16 in.) $\times \ d = 0.9d$, instead of d with regard to the cover and diameter of the bars.

$$\tau_{max} = \frac{V \frac{bd}{2} \frac{d}{4}}{\frac{b}{12} bd^3} = \frac{V}{b \times 0.67d} \qquad \text{(B1)}$$

and substitute M_r/a for V and find out the a/d value where $\tau_{max} = 0.1 f_c'$

$$0.1 f_c' = \tau_{max} = \frac{0.246 f_c' bd^2}{a \, b \, 0.67d} \ ; \ a = \frac{0.246 f_c' bd^2}{0.1 f_c' b \times 0.67d} = 3.68d \text{ and } a/d = 3.68$$

The result gives the important answer that beyond a ratio of 3.68 the beam will fail in bending before the principal stress above the support due to shear will reach the tensile resistance of the concrete.

Investigating Morrow's and Viest's Specimen B70B2 which has an a/d ratio 4.82, we get the following values for the bending moment acting on the beam and for the bending moment resistance of the beam as far as the load-bearing capacity of the compression zone is concerned.

$$M = 20000 \times 77 = 1{,}540{,}000 \text{ in.-lb}$$

$$M_r = 0.306 \times 2370 \times 12 \times 14.5^2 = 1{,}800{,}000 \text{ in.-lb}$$

The values show us that the beam could have failed in pure bending compression and if we bear in mind that the shear reduces the

*For full report see ACI-ASCE Committee 327, "Ultimate Strength Design," ACI JOURNAL, V. 27, No. 5, Jan. 1956 (*Proceedings* V. 52), pp. 505-524.

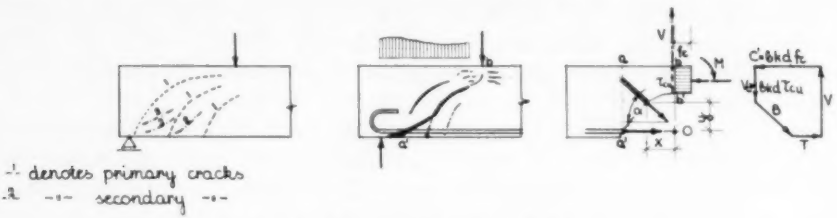


Fig. A—Free body of shear failure

resistance of the beam in bending as we shall see later, the compression failure of the beam in bending becomes apparent.

Specimen B84B4 having an a/d ratio of 5.80 broke under similar conditions. The bending moment acting on the beam was $M = 25000 \times 92.7 = 2{,}310{,}000$ in.-lb and the bending moment resistance limited at this time by the percentage of the main steel [$p = 3.294/(12 \times 14.5) = 0.019$] was $M_r = 14.5^2 \times 12 \times 0.019 \times 60{,}000\ (1 - 0.59 \times 0.019 \times 60000/3950) = 2{,}370{,}000$ in.-lb which proves again that the beam could have failed in pure bending.

Thus the writer is not particularly concerned by the low values of V_u/f_c' given in Table 1, if we take the values calculated by Eq. (B1) (tabulated in Table 1). Since the writer is convinced that reinforced concrete beams behave as homogeneous materials before the cracks occur the corrected figures give a better picture of the problem.

The writer is convinced that most of the specimens showing particularly low values of V_u/f_c' failed naturally in bending under a force, which gave a small nominal stress in shear, but it was large enough to break the beam in bending due to its big lever arm. The presence of diagonal cracks, which might form the wrong impression that the cause of failure was shear instead of bending is reasonable since we know from elementary statics that the whole lower part of a beam is in tension and that the largest stresses occur not on the vertical sections but along the lines of the stress trajectories.

It follows that it is advisable to analyze only the case of shear compression failure when the cross section is otherwise capable to balance the pure bending effect. (Splitting along the main steel can also give a low value of V_u/f_c' approaching 3.68, the limit value of a/d, as the specimen of Morrow's and Viest's test series, having an a/d ratio of 3.86, but the writer agrees with the authors that this can be avoided by having nominal amount of stirrups. Hence the writer is convinced that the code is right in specifying a limit of permissible shear stresses under which it is not necessary to investigate shear).

CONDITIONS OF EQUILIBRIUM

In the case of combined shear and bending shear stresses are also present in the compression zone. Among the test data Gyengo[51] shows that the shear stress in the compression zone is f_c/v_1, where $v = 5$. Menyhart[52] proves by theoretical consideration that according to Mohr's circle and taking 0.176 f_c' for τ_{cu}, the system can always be balanced by a compressive stress $f_{cp} \geqq 0.8\ f_c$ where f_{cp} denotes the direct stress acting parallel to the axis of the beam in the compression zone, and τ_{cu} denotes the shear stress in the compression zone. If the stresses exceed the values given there is a great possibility for shear-compression failure.

Another important difference between the authors' opinion and the writer's concerns the shear resistance of the main reinforcement. The writer does not agree that the shear resistance of the main bars should be taken into account in calculating the resistance in shear, since this resistance takes place only after large deformations. The writer's opinion is that a sufficient amount of links should stop the splitting effect at the level of tensile reinforcement, which is believed by the writer to occur in under-reinforced beams in the web. In such a case the structure is resisting the shifting effect of shear with its ultimate strength. This case is typical in the author's examples of Beam T 32 in Fig. 11 (among the unreinforced beams) and Beam 29 in Fig. 21 (among the reinforced beams). Since the inclination would also be reduced in beams properly reinforced by vertical stirrups and inclined reinforcement due to the spalling effect of the concrete at the tension zone (see Fig. C, left) the writer believes that the shear resistance of the main steel to be a certain safety factor but not large enough to be taken into account before excessive deformation of the beam has occurred.

Thus the writer will assume in the following that T, the force in the main reinforcement will act horizontally and supposing we cut out the case of $M > M_r$ and we deal only with the case of $M < M_r$ we are left with only two possible sources of shear failure, viz., (a) by compression and (b) tension.

(a) This failure takes place only when the shear reinforcement is not adequate to take its portion of the shear force and hence it leaves too large a part of the shear force which cannot be resisted by the compression zone. In this case the beam fails by the crushing of concrete of the compression zone along a diagonal plane which corresponds to

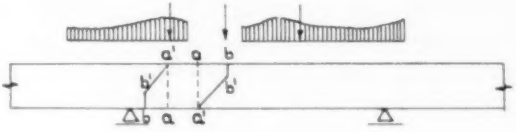


Fig. B—Shear distribution at various cross sections

the principal plane of the stresses of the compression zone after the redistribution of forces, and the shear reinforcement follows this strain by yielding. So this type of failure is a real shear failure.

(b) Shear tension failure is really a bending failure. We can explain its mechanism by analyzing the free body diagram in Fig. A. For every statical equilibrium three conditions must be satisfied, viz., $\Sigma V = 0$, $\Sigma H = 0$, and $\Sigma M = 0$. If the first condition is not satisfied shear-compression failure takes place and the two parts of the beam are sheared from each other. Because of this inadequate (amount or spacing) web reinforcement, the portion of shear for the compression zone becomes too large; in fact larger than can be adequately balanced and thus any excess of stresses in the compression zone would cause an immediate failure.

If we cut out the possibility of an under-designed compression zone we are only restricted to finding out kd (i.e., the extension or depth of the compression zone). The procedure for this is by equating the moments to zero and it is shown in the design part.

We are now only left with summing up the horizontal forces (obtained from the other two conditions) and to balance the unbalanced horizontal forces with T, the force in the main tensile reinforcement. Since in addition to the shear effect on the web reinforcement the value of T is of major importance, we will see the derivation of this value and the influence of shear on the resistance of the beam in bending and a method to plot the bending moment resistance diagram of the beam.

FORCES IN THE WEB AND MAIN REINFORCEMENT

The effect of bending (surcharge) on the web reinforcement was theoretically treated by Gyengo[51] showing surcharge on the web reinforcement intersecting in the middle of an inclined crack. Since the position of the inclined crack cannot be predicted, it is interesting to see the influence of the relative position of web reinforcement and the inclined crack on the value of the force induced in the web reinforcement. Limit cases for B_{max}, T_{min}, B_{min}, and T_{max} are among the infinite number of values that can be found.[53] B_{max} and T_{min} (Case 1) will occur if the web reinforcement is located at the outside edge of the inclined crack. B_{min} and T_{max} will occur if the web reinforcement is located at the inside edge (adjacent to the compression zone) of the inclined crack (Case 2). Eq. (13), (14), and (15) describe the two limit cases and are derived from the three equilibrium conditions of the free body diagram (Fig. A). Eq. (17) shows the maximum surcharge of bending on shear reinforcement and Eq. (15b) gives the maximum value of T, the force in the main reinforcement. Eq. (20) and (21) are of major

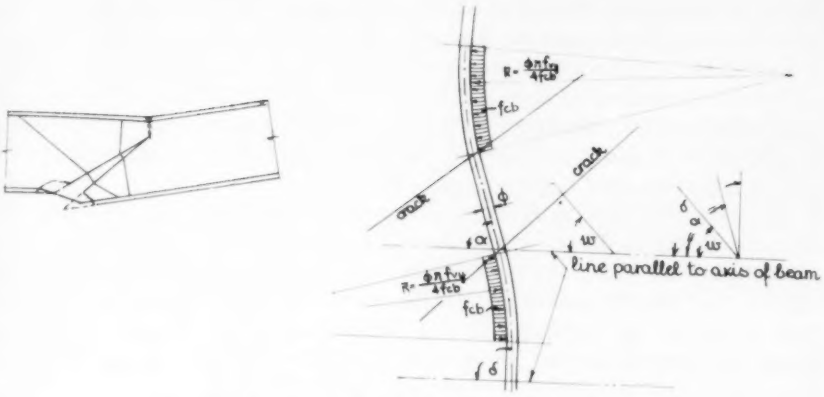


Fig. C—Spalling effect of concrete at tension zone

interest. They are derived for Case 2, to find out B_{min} and T_{max} in the case of vertical stirrups. The equations bear four important results:

(1) Eq. (20) shows that a sufficient amount of bending moment can provide the shear resistance against the whole shear force and no additional web reinforcement is required except the nominal amount of links.

(2) Eq. (21) shows that T and consequently k, the factor for the depth of the compression zone remain unchanged in comparison to those of simple bending. Hence in obtaining T and C' the shear resistance of the compression zone (to obtain this value it is only required to distribute C', the horizontal compression force by the friction or shear factor of the concrete which is $V_c = C'/v$) simple formulas of pure bending can be used. The equations show in a more general form that the force in the vertical stirrup is always the difference of the shear force and the shear resistance of the compression zone (i.e., $B = V - V_c$). The force in the main reinforcement is always $-C'$. In deriving the equations we used the equivalent rectangular stress block for simplicity, which is used by the British Code of Practice. We assumed that wherever there is a compression force there is a certain amount of friction or shear force present proportional to that compression force. Therefore the distribution of shear stresses follows the same distribution of compression stresses and so they are uniformly distributed on the compression zone.

(3) We can see from Fig. E1 that T occurs at a different vertical cross section $\overline{aa'}$ different from that of simple bending $\overline{bb'}$. That is because the cross sections no longer remain vertical to the axis of the beam but due to shear they become a cross section compounded of a flat surface (Line bb' on elevation) and a curved surface (Curve $b'a'$ on elevation) intersecting at a' with the main reinforcement (see Fig.

B). T is necessarily found to be at a different vertical cross section but with an equal and opposite force C. That simply means that the force T calculated by the assumptions of simple bending (i.e., assuming that fictitious vertical cross sections occur only in simple bending) is shifted by the horizontal projection of the inclined cracks towards the supports in the case of positive bending moments or towards midspan in the case of negative bending moments. This fact is significant in the design of a structure. It means that our present practice of following the bending moment diagram and bending up the main bars where they are not further necessary is not safe. To avoid tension failure we should extend the cut-off points of the bending moment resistance diagram with distance t as indicated in Fig. D (cut-off point denotes the points along the length of the beam where main bars can be curtailed or bent up).

(4) In the distribution of vertical stirrups the writer does not find the term $d(1-k)$ i.e., the maximum possible distance for vertical stirrups safe enough, because the inclined cracks become steeper towards midspan and supports. The cracks can be particularly steep under and in the neighborhood of a point load (e.g., the crack patterns of Bach and Graf, Specimens 1203 and 1205). In such cases the compression zone is not capable to counterbalance the shear force alone and the distance between vertical stirrups becomes too large. In fact far too large to attract the inclined cracks steeper than 45 deg. The compression zone will not be able to counterbalance the remaining shear forces and the beam will fail in shear like an unreinforced concrete beam, producing excessive cracks and a large deflection. The writer is convinced that the only safe solution is to follow the stress trajectories of the beam in question, or since this is impractical to follow the stress trajectories of similar beams from text books, and to provide stirrups at distances t from where they are necessary. It should therefore be noted that t^* is always smaller than $d(1-k)$ and the latter gives only correct distances at the supports of simply supported beams or at the points of contraflexure of continuous beams.

From the observations made for Case 2 we have reached the following important results for the case of vertical stirrups (and as we shall see, with a few additional observations we can reach similar conclusions for bent-up bars):

1. Factor k and thus the depth of the compression zone remains unchanged compared to the case of simple bending.

2. From 1 it follows that the maximum force in the main steel is the same as the value calculated by the rupture line theory of simple bending. Consequently the equations derived for simple bending remain unchanged.

*t denotes the horizontal projection of inclined cracks.

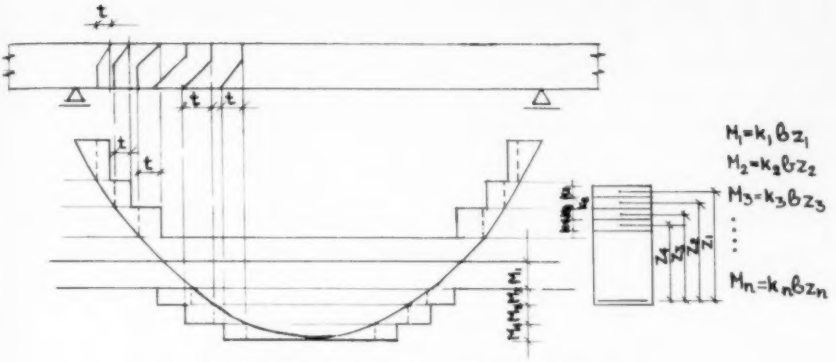


Fig. D—Bending moment resistance diagram

3. Due to the inclination of the cracks T_{max} at Section aa' is equal to the value computed for Section bb' by the theory of simple bending. To avoid tension failure we have to extend the points of the bending moment resistance diagram by a distance t towards the supports or midspan as shown in Fig. D.

4. Pull in stirrups = shear force — shear resistance of the compression zone (i.e., $B = V - V_c$). The writer's justification for this is that the inclination of the stirrups after yielding in the inclined crack is not as excessive as is shown in Fig. C but is due to the acute change in the direction of the steel and to the high bearing pressure induced by the relatively small curvature of the reinforcement ($1/R \approx \infty$). The steel must be balanced on a much larger radius and hence occupies an almost vertical slope in the inclined crack. It should also be noted from Fig. C that the slope of the main steel becomes much smaller, in fact almost horizontal, and thus the shear resistance of the main steel becomes much smaller too, due to the spalling of the concrete.

5. Spacing of the vertical stirrups should not be larger than the horizontal projection of the inclined cracks which at an inclination of 60 deg is $0.577d(1-k)$, or even less at steeper inclinations which can be seen in Beam 38 (Fig. 21), in Beams 38 and 47, and in the crack patterns of T 35 (Fig. 18). Similar patterns can be observed in the tests of Bach and Graf[54] wherever the distribution of the web reinforcement was reasonably adequate and where only fine crack patterns occurred. From Case 1 the writer reached the last important conclusion:

6. It is important for engineers to be aware of the surcharge induced in the vertical stirrups by bending but it must be ignored for practical reasons by a safely chose value of v. The engineer must also bear in mind that the main steel can counterbalance a certain amount of shear force (but not as much as was proposed by the authors and others)

without causing excessive cracks, horizontal splittings, or high deformations in the beam. The writer holds the view that ultimate load theories should not necessarily regard the failure of a structure simply by the appearance of a crack just as in the case of hooked or deformed bars. Failure should be regarded at a certain strain or formation of a major crack.

DESIGN OF MEMBERS SUBJECTED TO COMBINED SHEAR AND BENDING

The following simple procedure may be used for vertical stirrups:

Design beams as though they were only subjected to pure bending and neglecting shear forces. Secondly, find the shear resistance of the compression zone of the cross sections, and deduct this value from the given shear force. Furnish the cross section with the amount of stirrups required wherever the shear resistance of the compression zone proves to be insufficient.

For bending furnish the same amount of main reinforcement as obtained by the theory of simple bending. Extend the bending moment resistance diagram by the distances t because $d(1-k)$ can be assumed equal to t since for bending it is on the safe side. (The plotting of such a diagram is shown in Fig. D taking into account the formation of inclined cracks).

Struts subjected to thrust with combined bending and shear, or statically indeterminate structures where shrinkage stresses are considerably great, can be treated in a similar manner. First design the structure regardless of the shear forces for the eccentric thrust or pull by the well-known plastic theory, then find the values of kd and V_c, the depth and the shear resistance of the compression zone, respectively, and design the stirrups for the remaining shear force.

Determination of the shear reinforcement of whole shear span

For the case of a continuous beam loaded with uniformly distributed load the determination of the shear reinforcement of whole shear spans will be as following:

(a) Distribute the shear span into equal distances, and calculate each cross section separately for the given static conditions.

(b) Furnish each section with the amount of reinforcement for the worst case, i.e., use the largest area of steel on the smallest pitch obtained from the calculations made on the two boundaries of such a section until shear reinforcement is required. For example the boundaries of Section II are 1-1 and 2-2 as shown in Fig. E(b).

A typical scheme of shear reinforcement is shown in Fig. E(a), the stronger lines representing heavier shear reinforcement, and the thinner lines representing lighter reinforcement.

In Section II the area of each pair of stirrups is the largest because it is at Point O of the bending moment diagram, and no shear resistance is provided by the bending moment to the cross section. Hence the intensity of the shear force is still high. The pitch of shear reinforcement is, however, quite large since the inclination of the inclined crack is 45 deg. In Section I the area of each pair of stirrups is smaller than in the section mentioned before, because the rate of change of the bending moments is larger than the rate of change of shear forces at the support and thus the bending moment provides a larger shear resistance at the supports. The pitch is close, because it is controlled by the steeply inclined cracks in the vicinity of the supports. In Section III the area of stirrups is the smallest, due to the decrease of the shear forces and the increased shear resistance of the compression zone provided by the increasing bending moment.

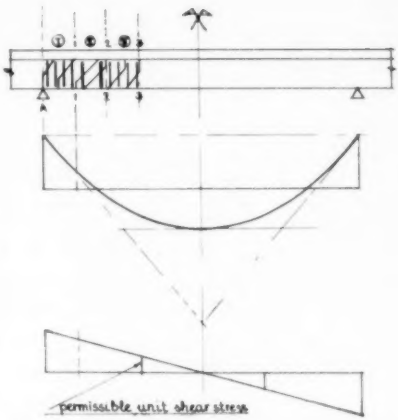


Fig. E(a)—Typical scheme of shear reinforcement

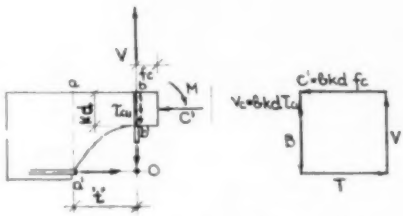


Fig. E(b)—Free body diagram of shear reinforcement

Design of structures with bent up bars

A glance at Fig. C proves that if the line of action of "B" intersects Point O, the following important conclusions can be made as in the case of vertical stirrups:

1. The presence of shear reinforcement does not alter the factor k and thus the depth of the compression zone calculated by ultimate theory remains the same.

2. At this position the inclined bar does not get any extra surcharge due to bending and the Force B in the bar is equal to (shear force—resistance to shear of the compression zone) $\times$ $1/\sin\alpha$.
In symbols

$$B = (V - V_c)\frac{1}{\sin\alpha} \qquad (11)$$

where α denotes the inclination of the main bar to the axis of the beam.

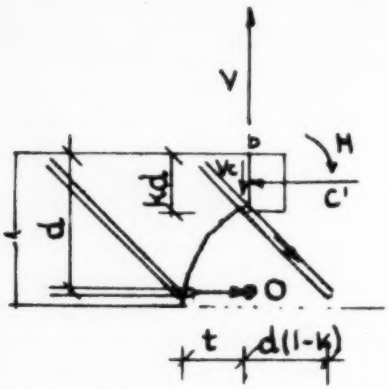


Fig. F—Maximum distance of inclined reinforcement

If $\alpha = 45$ deg Eq. (11) becomes

$$B = (V - V_c)\ 1.41 \quad \ldots (12)$$

This will lead to an important simplification.

3. As opposed to the case of vertical stirrups the intensity of Force T is smaller than that calculated for the case of simple bending, because the horizontal component of Force B reduces T (see Fig. A). This follows logically from the sum of the horizontal forces. Thus the extension of the bending moment diagram by t distances is not necessary.

4. Since the spacing of the inclined reinforcement is controlled by the horizontal projection of the inclined cracks, the writer does not agree that the Formula $2(d - kd)$ given by the authors (pp. 448, 449) is safe because the inclined cracks follow the stress trajectories and with the exception of the supports or points of contraflexure they are always steeper than 45 deg. This can be seen on the crack patterns of Beam 38 (Fig. 21), or Beam 47 (Fig. 23).

The cracks can be particularly steep in the case of concentrated forces (see for example Bach's and Graf's Beams 1203 and 1205 on p. 458).

From Fig. F, the maximum distance of the inclined reinforcement should not be more than $t + d(1 - k)$, which for the case of an in-

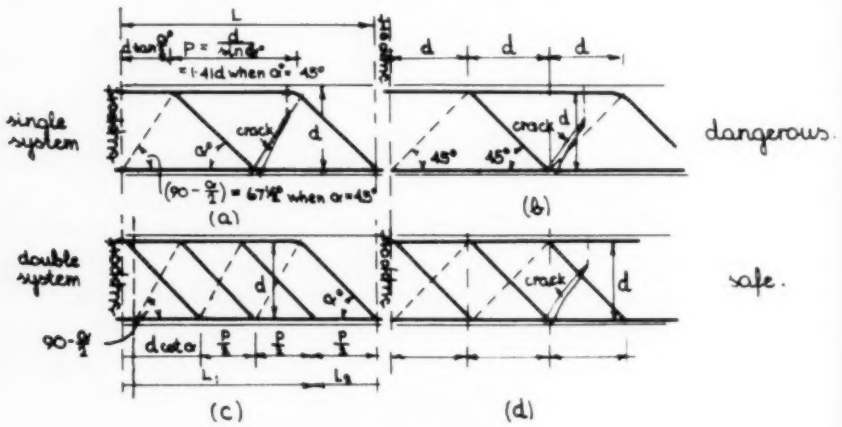
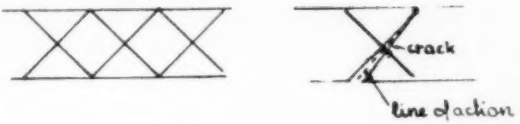


Fig. G—Systems of shear reinforcement

Fig. H—Assumed and real lattice action



clined crack of 60 deg will be $0.577(d-kd) + 1(d-kd) = 1.577(d-kd)$. This expression is much smaller than the one given by the authors.

The writer fully agrees with the authors that the so-called single system, commonly used in English practice, is not safe but is in fact highly dangerous. But similar solutions can be found in the practice of almost every country. The writer has shown in Fig. G four commonly used systems, two of which are not safe, because the inclined bars cannot intersect an inclined crack of 45 deg or steeper.

The basic mistake of such practices is that they assume that two web members of an imaginary lattice system are always acting simultaneously together, viz., the concrete and the web reinforcement. In fact there is only one, i.e., the inclined steel. The inclined cracks split the concrete along the assumed line of action of the fictitious strut (see Fig. H) leaving the steel alone to counterbalance the shear force.

The writer has worked out a practical example according to the British Code of Practice.[34]

NUMERICAL EXAMPLE

Let us design a 13 x 25½-in. beam having a 16-ft span loaded by a uniformly distributed load of 2.75 kips per ft and by two point loads of 22 kips acting at quarter span. The nominal mix of concrete being 1:2:4, the permissible stress for the load factor method in bending will be: 2/3 x 1000 lb = 666 psi assumed to act uniformly in the compression zone. The permissible stress for steel having a guaranteed yield stress of 60,000 psi will be 30,000 psi in bending and 20,000 psi in shear. The total load on the beam will be $W = 88$ kips and the reactions (A) equal to 44 kips. The maximum moment at midspan will be $M_{max} = 44 \times 8 - 2.75 \times 8^2/2 - 22 \times 4 = 176$ ft-kips = 2,120,000 in.-lb.

Now using the design tables[55] for the case of simple bending, we get $Q = 2{,}120{,}000/(13 \times 25.5^2) = 250$ and for the area of main reinforcement $A_m = (0.5 \times 666 \times 13 \times 25.5)/30{,}000 = 3.72$ sq in. For furnishing the area required we will put five 1 in. diameter bars giving a total area of 3.93 sq in. in the bottom layer of the beam.

Since the stresses of the shear force diagram shown in Fig. I are higher than permissible we have to provide a sufficient amount of shear reinforcement to carry the total amount of shear force where the excess occurs.

At Section 1-1 the value of the bending moment and the shear force is $M_1 = 44 \times 4 - 2.75 \times 4^2/2 = 154$ ft-kips $\approx$ 1,850,000 in.-lb; $V_1 = 44 - 4 \times 2.75 = 33$ kips. Factor k of the compression zone is taken from the tables instead of using Eq. (14c). Thus $Q = 1{,}850{,}000/13 \times 25.5^2 = 219.5$, where $k_1 = 0.4155$ and $C_1' = 0.4155 \times 25.5 \times 13 \times 666 = 92{,}000$ lb. The shear resistance of the compression zone will be $Vc_1 = C_1'/v = 920{,}000/5 = 18{,}400$ lb. The area of inclined reinforcement is

$$A_{B_1} = \frac{(V_1 - V_{c_1})\ 1.41}{p_w} = \frac{33{,}000 - 18{,}400}{20{,}000} \times 1.41 = 1.035 \text{ sq in.}$$

and the bending moment, shear force, and factor k for Section 2-2 will lb. The shear resistance of the compression zone will be $Vc_1 = C_1'/v = 44 - 2 \times 2.75 = 38.5$ kips; $Q_2 = 990{,}000/(13 \times 25.5^2) = 117$; $K_2 = 0.1945$; and $C_2' = 0.1945 \times 25.5 \times 13 \times 666 = 43{,}000$ lb. Similarly in Case 1-1 the compression force and the shear resistance of the compression zone will be $V_{c2} = C_2'/v = 43{,}000/5 = 8{,}600$ lb. The area of inclined reinforcement

$$A_{B_2} = \frac{(V_2 - V_{c_2})\ 1.41}{p_w} = \frac{38{,}500 - 8{,}600}{20{,}000} \times 1.41 = 2.12 \text{ sq in.}$$

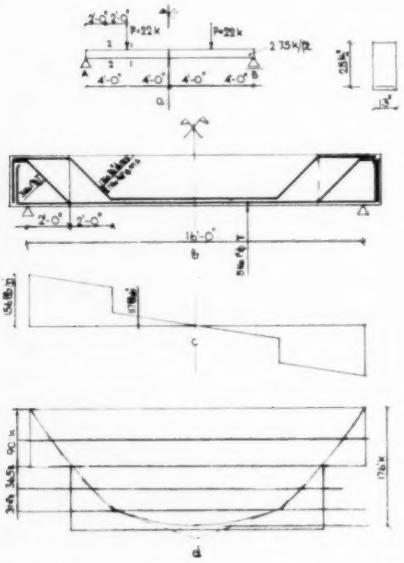


Fig. I—Shear and bending moment diagrams according to Fig. D

Thus the areas of main and shear reinforcement being known we have to plot only the bending moment resistance diagram to make sure that the main bars can be bent up in the places where they are necessary for shear. Because of the heavy shear the area of main reinforcement is not sufficient to provide sufficient amount of shear reinforcement we had to provide one 7/8 in. and one 3/4 in. diameter mild steel bars in the second layer as extra shear reinforcement for Section 1-1. As the bars would almost touch each other in the middle of the beam the writer preferred to keep the bars in one piece although one cannot make use of this fact in bending. The reinforcement diagram of the beam is shown in Fig. I.

THEORETICAL INVESTIGATIONS

A view of Fig. A explains at once the equilibrium of forces. If $Q_m > bd\tau_u$ the three basic equilibrium conditions can be expressed as

$$V = bkd\ \tau_{cu} + B \sin\alpha \quad \text{.....(13)}$$

$$M = bkd\, f_c \left(d - \frac{kd}{2}\right) + B \sin\alpha\, x - B \cos\alpha\, y \quad \text{.....(14)}$$

$$T = bkd\, f_c - B \cos\alpha \quad \text{.....(15)}$$

Case 1—Maximum force in shear reinforcement and minimum force in longitudinal (main) reinforcement if the position of shear reinforcement is at the lower edge of the inclined crack: Substituting $y = 0$ and $x = t$ into Eq. (13), (14), and (15) we get

$$V = bkd\ \tau_{cu} + B \sin\alpha \quad \text{.....(13b)}$$

$$M = bkd\ f_c \left(d - \frac{kd}{2}\right) + B \sin\alpha\, t \quad \text{.....(14b)}$$

$$B_{max} \sin\alpha = \frac{2nV - vd + t + \sqrt{(-2nV + vd - t)^2 + 4n\,(vdV - nV^2 - M)}}{2n} \quad (16)$$

And after a minor transformation

$$B_{max} = \frac{2nV - vd + t + \sqrt{v^2d^2 - 4nM + 4nVt - 2vdt + t^2}}{2n \sin\alpha} \quad (17)$$

Case 2—Minimum force in stirrups (or inclined bar) and maximum force in longitudinal (main) reinforcement: The position of stirrups (inclined reinforcement) is at the upper edge of the inclined crack. Again we have

$$V = bkd\tau_{cu} + B\sin\alpha \quad \text{.....(13c)}$$

$$M = bkd\, f_c \left(d - \frac{kd}{2}\right) - B\cos\alpha(d - kd) \quad \text{.....(14c)}$$

where

$$k = \frac{V - B \sin\alpha}{db\tau_{cu}}$$

and substituting the value for k

$$M = v(V - B \sin\alpha)\left(d - \frac{V - B \sin\alpha}{2b\ \tau_{cu}}\right) - B \cos\alpha \left(d - \frac{V - B \sin\alpha}{b\ \tau_{cu}}\right)$$

from where as in Eq. (17), B_{min} can be expressed thus

$$B_{min} = \left\{ 2nV \left(\sin\alpha + \frac{1}{v}\cos\alpha\right) - d\cos\alpha - vd\sin\alpha + \left[V^2 \frac{4n^2}{v^2} \cos^2\alpha \right.\right.$$

$$\left. - V\frac{4nd}{v}\cos^2\alpha - M\left(4n\sin^2\alpha + \frac{8n}{v}\sin\alpha\cos\alpha\right)\right]^{1/2} + d^2\cos^2\alpha - v^2d^2\sin^2\alpha$$

$$\left. + 2vd^2\sin\alpha\cos\alpha \right\} \Big/ \left(2n\sin^2\alpha + \frac{4n}{v}\sin\alpha\cos\alpha\right) \quad \text{.....(17a)}$$

and

$$T_{max} = bdf_c \frac{V - B \sin \alpha}{bd \tau_{cu}} - B \cos \alpha = v(V - B \sin \alpha) - B \cos \alpha \quad (15b)$$

and in the case of vertical stirrups $T_{max} = v(V - B)$.

Investigation of Case 2 for the case of vertical stirrups

For case of vertical stirrups we could use Eq. (17a) and (15b) for computing T_{max}. However, the equations give exactly the same answer for T_{max} as the well-known and simple equations

$$M = bkd\, f_c \left(d - \frac{kd}{2} \right) \quad (14c)$$

$$T = bkdf_c \quad (15c)$$

which were derived for the case of pure bending.

Let us take the case of simple bending and choose for k a value such that the compression zone should have a shear resistance equal to V. Then

$$kbd \frac{f_c}{v} = V \quad \text{and} \quad K = \frac{v\,V}{bd\,f_c}$$

Now substituting this value of k in Eq. (14c) we get

$$M = vVd - \frac{v^2V^2}{2b\,f_c} \quad (18)$$

and

$$T = \frac{v\,V}{bd\,f_c} f_c bd = vV$$

Now take the case of combined shear and bending. Let us take the same bending moment as before, i.e., Eq. (18) and substitute it in Eq. (19) which is the implicit form of Eq. (17a).

$$B_{min} = \frac{2nV - vd + \sqrt{(-2nV + vd)^2 + 4n(vdV - nV^2 - M)}}{2n} \quad (19)$$

Now after the substitution and minor transformation we find

$$B_{min} = \frac{2nV - vd + \sqrt{(-2nV + vd)^2 + 4nVvd - 4n^2V^2 - 4nVvd + \frac{4n\,v^2V^2}{2b\,f_c}}}{2n}$$

$$B_{min} = \frac{2nV - vd + \sqrt{(-2nV + vd)^2}}{2n} = 0 \quad (20)$$

This means that no additional Force B should be applied because the whole shear force is taken by the shear resistance of the compression zone. Similarly $T_{max} = v(V - B)$. This is exactly the same answer that we obtained before, which means this is the limit case of pure bending and bending combined with shear, from where shear reinforcement should be provided.

Now for the other limit case where $M = 0$ from Eq. (19) we get the following equation

$$B_{min} = \frac{2nV - vd + \sqrt{4n^2V^2 - 4nVvd + v^2d^2 - 4n^2V^2 + 4nVvd}}{2n} = V \quad (21)$$

And so on, we can prove in any other cases of

$$\frac{M}{V} < vd - \frac{v^2V}{2b f_c}, \quad \frac{M}{V} > vd - \frac{v^2V}{2b f_c}$$

the compression zone balances the shear force alone) that T (and of course the lever arm) is equal to the values calculated by the equations of simple bending.

That means that the same simple way can be used for calculating the main steel or plotting the bending moment resistance diagram as in the case of simple bending but the points of the bending moment resistance diagram should be extended by the horizontal projection of the inclined cracks accordingly.

In the case of shear reinforcement (as we have seen in the previous examples) we get the following simple procedure: Deduct from the shear force the shear resistance V_c of the compression zone. For finding the value of V_c distribute the compressive force C' use the shear v or friction factor of concrete. Then take the remaining shearing force by stirrups.

Thus

$$C' = -T = v\,V_c;\; V_c = \frac{-T}{v} = \frac{C'}{v}$$

The area A_{st} of stirrups will be

$$A_{st} = \frac{V - V_c}{2 f_v} \quad (22)$$

where f_v represents the yield stress of the web reinforcement.

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By PHIL M. FERGUSON*

Part 1 of this series of papers devotes some emphasis to a discussion of tests made by the writer (using secondary cross beams for loading devices or for supports or for both) and concludes that the "effects of vertical restraints at load points are believed to be limited to beams with very low a/d ratio." This conclusion makes it necessary for the writer to review some of the authors' treatment of this subject to point out certain aspects of their supposedly similar specimens which were influential in increasing the shear strengths which they obtained.

In both the authors' beam and those of this writer the clear distance between the secondary loading beam and the reaction beam was small. The authors used a clear space of 8.5 in. with an effective depth d apparently about 7 in.; this writer used a clear space of 9.5 in. and an effective d of 10 in. Thus, neither group of beams had room for a diagonal crack to form without (1) crossing through the loading beam, or (2) crossing through the reaction beam, or (3) intersecting the loading beam in such a way as to develop into a crushing shear-compression type failure. Hence, in assessing the diagonal tension strength, one must consider the strength of these cross members as a part of the system, acting much like oversize local stiffening and strengthening units.

Under these conditions it is important in an investigation of the main beam strength that neither cross member be heavy compared to the main beam. In this respect the authors' end beam with its 7 in. thickness adds a massive "end anchorage" against the formation of any diagonal crack. The loading cross beams are more appropriately sized and it is unfortunate that the end beams were not more nearly the same size.

It is also obvious when one looks at the plan view of the authors' beams (see Fig. 13) that their experiments did not really deal with a beam *without* stirrups, but rather with a beam heavily stirruped near the reaction end and near the load point and lacking in stirrups only for a 9 or 10 in. length between these cross beams. Thus any diagonal

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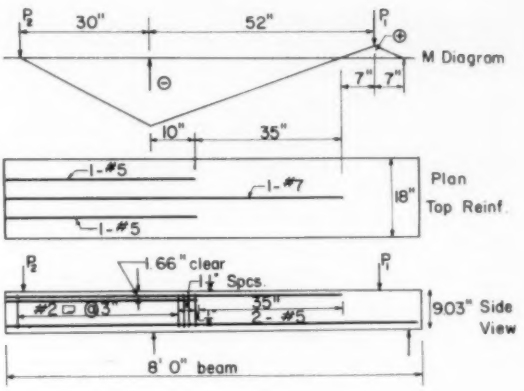

Fig. J—Details of Beam B42

crack which tended to break through the reaction beam encountered heavy stirrups; likewise if it tended to break out through the loading beam it found stirrup restraint. It is not surprising that their results failed to confirm those the writer had found earlier using beams without stirrups.

Nor is it surprising that several of the authors' beams failed toward the supported end which was without the cross beam at the reaction. There was simply more clear web length there which was free from stirrups. It appears that Beam 8 failed through the loading cross beam only by cracking high enough to go *over* the loading beam stirrups. It is interesting to note that while the authors were (possibly) correct in stating that Beam 8 "failed near the support," the failure was not *across* the support. At the support the vertical compression stresses were adequate to clamp the bars and prevent their stripping out. Except for the use of plain bars, the hooks would probably have not been needed there.

The authors point out in connection with the writer's tests that the restraining effect of the load on top of the beam would not be "likely to extend along the beam more than a distance equal to half its effective depth, or possibly even less." This is a reasonable guess. However, their "confirmation" seems to overlook the fact that no diagonal tension crack ever continues at a nearly 45 degree slope to the compression face of a beam unless something forces it upward as the only route to collapse. A diagonal crack, even with such small a/d ratios as 1.35 and 2.09, flattens out in the flexural compression zone and thus does come under the influence of vertical compression when the load rests directly on the top of the beam. The authors quote at length about longitudinal strain measurements on the top surface of a beam, but the implications of this are lost on this writer. (Diagonal cracks do not reach the top surface of a beam except at distances from the reaction in excess of three to four times the effective depth of the beam.)

An excellently planned series of beams with cross beam loading and spread reactions has recently been reported.* This is a more complete study than that made by the writer and it is quite valuable in defining the range within which the vertical compression stresses below a load or over a reaction may be expected to influence beam shear strength. This series shows that in a beam 12 in. deep some increased shear strength from loads and reactions applied directly to the beam occurs when the load is as far as 42 in. from the support, and possibly when as far as 48 in. Of course, the percentage effect decreases rapidly with increasing a/d. Taylor concludes that the first diagonal cracking load is essentially the ultimate load when the loads and reactions are applied through cross beams.

In their conclusions for Part 1 the authors emphasize "that a good connection between the main beam and cross beams is essential." If each end of a 4 in. wide cross beam (without stirrups) cannot bring in *half* as much load as can be carried by a 4 in. main beam (also without stirrups), something is wrong with the quality of the cross beam. Certainly in the tests reported by the writer in 1956 no cross beam ever indicated the least sign of distress in either shear or flexure. The cross beams were in fact grossly overstrength even without the addition of stirrups.

The evidence the authors have marshalled (in Part 5) to indicate that their shear-tension failure is truly the result of diagonal tension, distinct and apart from bond stress, is interesting. For the cases studied the writer is inclined to accept this analysis at face value. In a way it confirms interrelationships between bond and shear stress that have been suggested earlier.†

However, the authors have extrapolated their information too far in their two final paragraphs of Part 5 where they state too definitely that bond stress is never a primary cause of failure; also that the calculation of bond stresses in a beam is unnecessary. Some of these statements seem to imply that they may need to be considered with limitations, but as they are stated any such limitations are too vague to be helpful. The end result could be dangerous.

Bond stress calculation in some fashion is certainly still significant in the case of lap splices in beams, or in the case of the lap of the main steel alongside the dowel bars in the stem of a cantilever retaining wall.

Even in a beam without splices, bond can be critical at places where shear stress is not particularly high. Fig. J. shows the dimensions and steel arrangement of Beam B42 which was tested at the University of Texas and which did fail in bond. The evidence in Fig. K is conclusive

*Taylor, R., "Some Shear Tests in Reinforced Concrete Beams Without Shear Reinforcement," *Magazine of Concrete Research* (London), V. 12, No. 36, Nov. 1960, pp. 145-154.

†Ferguson, Phil M.; Turpin, Robert D.; Thompson, J. Neils, "Minimum Bar Spacing as a Function of Bond and Shear Strength," ACI JOURNAL, V. 25, No. 10, June 1954 (*Proceedings* 50), pp. 881-884.

Fig. K — Bond failure of Beam B42

that this beam certainly did not fail in shear in the sense of diagonal tension. The beam reached its ultimate strength at a unit shear of 86 psi, a unit bond stress of 550 psi, and a calculated steel stress of 88.5 ksi. The concrete cylinder strength was 2950 psi. The deformed steel bars had no real yield point, but the bar stress at an offset of 0.002 in. was about 87.5 ksi and the ultimate 103 ksi.

It thus appears there are cases where one must calculate bond stress; and bond stress can be the critical stress. Until someone properly defines the limitation of that statement, designers must continue to calculate bond stresses or development lengths or in some manner limit the design bond conditions in their beams.

By K. HAJNAL-KÓNYI*

The paper contains much valuable information and many sound suggestions. In particular, the writer would like to endorse the authors' recommendation "that web reinforcement should generally be provided throughout the beam and not only in the zone of the maximum shear." (p. 457). Further, the authors' conclusion that "when deformed bars are used, the provision of stirrups is *essential*" (p. 720), is an improvement on a statement by the second author in a previous paper[56] "that stirrups are *valuable*† in preserving the integrity of beams with deformed steel."

The harmful effect of cutting or bending up the reinforcement with the diminution of the bending moment (p. 332) was clearly demonstrated by Emperger.[57] Referring to Fig. 16, the force in the tensile reinforcement at Section 2-2 is considerably greater than the tensile force corresponding to the bending moment at that section. The new French code[58] includes a clause to this effect (Clause 4,23511).

The authors' contention that "the use of the truss analogy for the calculation of stirrup size and spacing leads to incorrect results" (p. 326) was proved long ago, particularly in the discussion of the German Code of 1925. This code introduced the requirement of "full shear reinforcement" when the calculated shearing stress on the concrete exceeds 57 psi. Many engineers found this innovation unjustified. Reference is made to the writer's contribution[59] in which the beams included in Table 15 and the beams tested by Saliger and Probst were

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†Writer's italics

analyzed. The discussion in 1926-1927 was confined to beams with mild steel. How far the conclusions can be applied to shallow beams with steel of high tensile strength has yet to be investigated.

However, it has to be pointed out that in some of the beams listed in Table 9 on which the authors' conclusions are based, failure was due to slipping of the stirrups because their anchorage in the compression zone was inadequate. This was observed by Mörsch who suggested that rectangular stirrups would have probably given better results than the open ones.

The great importance of the efficient anchorage of the stirrups is not mentioned by the authors. It was emphasized by the writer in a previous discussion.[60]

It is regrettable that the authors' idea "that in normal beams bond failure is never a primary cause of failure of a reinforced concrete member subjected to both flexure and shear but merely a consequence of the redistribution of internal forces following the widening of the diagonal tension crack" (pp. 729-730) leads to the conclusion "that the calculation of bond stresses in a beam is not necessary, provided the beam is efficiently reinforced against shear failure."

This conclusion needs a few comments. Diagonal tension cracks develop and widen also in beams which are "efficiently reinforced against shear failure" and once this stage is reached the possibility of a further increase of the load depends on the efficiency of the anchorage. Many examples could be quoted but it may be sufficient to quote the following sentence from the authors' paper (p. 526) with reference to Walther's tests: "The beams with polished round bars (with hooks) failed in bond, the beam ends being destroyed by the hooks, but all the other beams failed in flexure-tension."

By disregarding the criterion of limiting bond stresses, the authors arrive at an incorrect conclusion regarding the factor of safety of Beam b in Table 9 (pp. 327-328). The writer was unable to find data about the cube strength of the concrete of this beam but since it was tested before 1911, an assumed cube strength of less than 3750 psi seems to be reasonable. With this assumption, the permissible local bond stress according to the British Code of Practice is 180 psi. With an assumed lever arm of 12 in. the permissible shear is $S = 2 \times 1.57 \times 3.14 \times 180 \times 12 = 21.3$ kips. Allowing 400 lb for the shear due to the dead weight of the beam, the factor of safety is $94.4/(2 \times 20.9) = 2.26$ as against only 1.65 according to the authors. As stated before, the anchorage of the stirrups in this beam was inadequate. The factor of safety would have been probably higher if slipping of the stirrups had been prevented by better anchorage.

The Authors' comments on p. 726 regarding the failure of Beam 28 in Table 31 in connection with the German Code are also fallacious

since tensile reinforcement consisting of plain bars without hooks as in Beam 28 is not permissible. The limit of 1.02 in. below which the bond stress need not be considered is based on the assumption that hooks are provided.

With regard to the remaining beams in Table 31 it should be borne in mind that in beams in which the shear is covered by bent up bars, according to the German Code only one half of the shearing force has to be taken into account in Eq. (10). In this way the values for u_u are reduced to one half of the values in the table and the discrepancy between Beam 28 and the four other beams listed in the table disappears. It is obviously absurd to relate the whole shearing force to the few remaining straight bars where the bar pattern is like that of Beams 35 and 37 to 39. The reduction to one half is a rough approximation, suitable for design purposes.

Quite apart from numerical values of the bond stresses it is clear from the mode of failure described in the last column of Table 31 that the full yield strength of the tensile reinforcement could not be developed in any of the beams in question. Indeed, once the stage represented in Fig. 6 is reached, the beam is "hanging on the hooks" which in these beams did not exist. This stage can be reached also in beams with adequate shear reinforcement as demonstrated by Beams 35 and 37 to 39. The proper conclusion to be drawn from Table 31 is that hooks must be provided on plain bars. The well-known fact that improved shear reinforcement delays the occurrence of bond failure does not mean that by this means alone bond failure can be avoided altogether.

The tests summarized in Table 30 have only proved that the resistance of bars to bond does not depend on the bond stress alone as obtained from Eq. (10) but also on the shearing stress in the concrete as has been pointed out by Mörsch. The logical conclusion would be to reduce the permissible bond stress with increasing shearing stress on the concrete.

The tests by Bernander have only proved that failure in diagonal tension is possible without bond failure and that beams without shear reinforcement should not be permissible.

It follows from the preceding considerations that the authors' conclusion regarding bond failure as quoted above is not justified. How can the tensile stress in a bar be developed without bond and/or anchorage? Even if it were generally true that bond failure was only the consequence of primary failure in shear, it would still be necessary to prevent this "secondary" failure by good design and detailing, i.e., by a safe anchorage of the straight bars.

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By A. HELFGOT*

I have read the entire series of papers dealing with shear in reinforced concrete beams with much interest. The results of the investigation are an important contribution to the subject.

Recently the Laboratory for the testing of materials and technological investigations of the Public Works Ministry of the Province of Buenos Aires (LEMIT) carried an investigation (subsequently published)† which might shed some more light on the behavior of concrete reinforced with bars devoid of the "traditional" hooks.

The object of the Buenos Aires investigation was to compare the behavior of concrete beams with tensile and compressive reinforcement. Beams were of rectangular cross section 3.30 m long submitted to two equal charges at the third points. Two kinds of steel were used as reinforcement: round mild common steel bars, with hooks at their ends and cold twisted bars with two longitudinal ribs on opposite sides, without end hooks. The longitudinal reinforcement consisted of straight bars all over the beams; shear stresses were absorbed by stirrups of mild 6 mm diameter steel in the first group of beams and of a special 4.2 mm diameter bar with a roughened surface to increase bond with the concrete in the second group.

The tests included data on flexural moments, deflections, as well as steel stresses corresponding to different stages of cracking especially as related to bond and shear.

The results indicate that end hooks in conveniently deformed bars are quite unnecessary and may in fact be highly injurious to the concrete. Furthermore, German specifications‡ now stipulate minimum anchorage lengths for deformed bars without end hooks. This undoubtedly is a major step towards the simplification of reinforced concrete structures.

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*Civil Engineer, Buenos Aires, Argentina.

†Helfgot, A., "Vigas de hormigon con armadura y comprimida," *Construcciones* (Buenos Aires), No. 153 and 154, pp. 1-22. See also Helfgot, A., "Poutres en bèton armées en compression et en fraction," *Béton Armé* (Paris), No. 21 and 22.

‡*Bestimmungen des Deutschen Ausschusses für Stahlbeton*, edited by Verlag von Wilhelm Ernst und Sohn, Berlin, 1955, p. 276.

By C. J. POSEY*

The authors give a good picture of the role of end anchorage in relation to web reinforcement. There is, however, evidence that some of their conclusions with regard to the action of hooks are not correct.†

If a cracked beam is to carry an appreciable load by acting like a two-hinged arch, the end anchorage must pick up its load with little slip. When the slip at each end plus the stretch in the length of the main bars permits too much distortion of the geometry, failure will result. This is why the ratio of load to slip, which might well be termed the "stiffness" of the anchorage, is of great importance. The most common types of pull-out tests do not take this stiffness into account, while beam tests do so only implicitly, with the likelihood of other effects obscuring comparisons.

The pull-out specimen used in the tests referred to was designed by the writer to (1) permit measurement of the load-slip ratio, with slip measured close to the loaded end, and to (2) have a stress distribution in the vicinity of the hook nearly identical with that in a beam (see Fig. L). The first series of tests showed enclosure of the hooks in spiral reinforcement does nothing more than prevent the concrete from splitting apart after it has cracked, and after the anchorage has slipped so far that its value as end anchorage in a beam would be nearly spent. The difficulty of getting good quality concrete inside the spiral insures that the load carried by a spiral-enclosed hook at the significant values of slip is less than that carried by the same hook without the spiral.

The writer's tests indicated that splitting in the plane of the hook will not occur if the ratio of the centerline diameter of the hook to

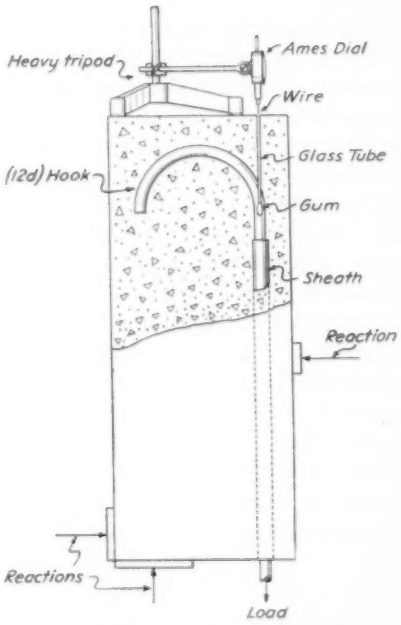

Fig. L—Pull-out specimen with beam-like stress distribution and device for measuring slip at loaded end of hook

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†Mylrea, T. D., "The Carrying Capacity of Semicircular Hooks," *Proceedings*, ACI, V. 24, 1928, pp. 240-272. (The writer would like to call attention to an error: the first word of the third line of p. 271 should be "splitting," not "slipping.") See also "Tests of Anchorages for Reinforcing Bars," by Chesley J. Posey, *Bulletin* 3, University of Iowa Studies in Engineering, Apr. 1933. Summarized in *Engineering* (London), V. 136, p. 217, Sept. 1, 1933.

the diameter of the bar is in the neighborhood of 12. (The author's $k = 11$) For ratios of 10, splitting sometimes occurs. It seems that the k value beyond which splitting will not occur should be a function of the cover. However, the value of $12d$ for a non-splitting hook diameter is in fairly good agreement with the value of $7d$ for radius of non-splitting bent-up bars in beams. What is more important, however, is that the load-slip ratio or stiffness of the anchorage increases with the diameter of the hook. This is true even when the embedded length is kept constant. Contrary to the authors' expectations, lengthening the straight portion of the bar beyond the hook does not improve its value as an anchorage.

By E. M. RENSAA*

The paper in five parts dealing with shear in reinforced concrete beams contains a great deal of information of interest to designers of reinforced concrete structures. It describes results of tests carried out both in Europe and America and also some done by the authors. The collection of data on an international basis is commendable and should be the rule for all research. It is evident that the authors have had to do a tremendous amount of work to collect and evaluate information on shear tests. The conclusions based on the study of such material seem generally to be well founded and there is little with which the writer disagrees. The following discussion does not necessarily indicate difference of opinion, but is intended to complement some of the information given in the paper.

The original conception of truss analogy assumed the concrete "compression diagonal" to have a slope of 45 deg corresponding to the line of principal stresses at the neutral axis. It was assumed that the line of compressive stress would continue on this angle down to the reinforcing steel. That was not correct where in addtion to shear there is also a bending moment which will modify the average slope of the "compression diagonal." This is illustrated in a paper by the writer.[61] The truss analogy is in the writer's opinion an excellent method of indicating the function of web reinforcement and it gives quite accurate results provided realistic slopes of the "compression diagonals" are being used.

The statement on p. 195 "that according to the various codes the shearing strength of a T- or L-beam is taken to be no greater than that of a rectangular beam whose breadth is equal to the breadth of the web of the T-beam" needs some clarification. If the elastic theory is used, it is usually found that jd for a T-beam is greater than for a rectangular beam of the same depth. The calculated shear stress will, therefore, generally be smaller for a T-beam than for a rectangular

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beam and the corresponding web reinforcement less. Apart from this inequality there are admittedly some other conditions which are favorable to T-beams. The principal advantage is its greater security against shear compression failure. A further advantage is that plastic deformation due to bending will not generally decrease the lever arm between the compression and tension resultants for a T-beam to the same extent as for a rectangular beam. This effect may also be noticeable before a crack has developed and will contribute to raising the cracking load of a T-beam above that of a rectangular beam. After a crack has developed and as the compression concrete is being increasingly stressed, the compression resultant will move downwards due to plastic deformation and this condition will continue until the steel starts to yield. When that happens the concrete compression resultant will again raise with an increase in the internal level arm until the compression concrete fails. With a relatively thin and wide flange the concrete compression resultant has less opportunity to move up and down and the internal lever arm will stay nearly constant at its maximum value. It would for this reason seem to be quite proper for the purpose of calculating shearing stresses to use a reasonably larger internal lever arm jd for T-beams than for rectangular beams with the same total depth.

These are conditions whose effect it should be possible to evaluate. It appears, however, to be quite unjustified to assume that T-beams have some unexplainable built-in immunity against shear failure. That is in effect what may be inferred from the ACI Code, Section 801(e) where beams with "T-beam action" are exempted from the shear provisions for the negative bending moment portion. It appears out of order to assume T-beam action where the flange is on the tension side and is supposed to crack and be inactive as part of the beam as is always assumed for bending. The compression resultant will act on the stem only and this is subjected to shear compression failure just as much as if the original beam was rectangular.

In practice the shearing stress in the stem of a T-beam is usually higher than in a rectangular beam of the same depth. While the flanges in most cases give security against shear-compression failure, there is an exception to this rule in cases where the strength of the flange concrete is used to the full limit. That may happen for long spans with close spacing of beams and a relatively thin flange. In such cases there may be even less security against failure than for rectangular beams, because of the relatively small area of concrete in the stem available for taking additional compression due to plastic deformation. The writer thinks that with proper allowance for the difference in the length of the internal lever arm, jd, there is no valid reason for exempting T-beams from the general rules of shear reinforcement. No such exemp-

tion is made in the reinforced concrete specification of the 1960 edition of the Canadian National Building Code. If it is considered to be essential to give T-beams a special advantage because of the greater shear-compression value it would then also be necessary to state the limits within which this could be done. Otherwise a trap is set for those who deal with unusual cases and follow code provisions to the letter.

Again on p. 196 it is criticized that codes do not take into consideration the transverse resistance of stirrups and main tension reinforcement. To this it might be said that it has been known from almost the beginning of this century that such effects exist. These effects are, however, of secondary importance and may not be present at time of failure. Furthermore, it has been pointed out by Mörsch[61] in several instances that a theory need not be exact provided the nominal stresses based on such a theory give correct and consistent safety factors. Mörsch had from the beginning of his work of developing the shear theory pointed out that the shearing stresses found by the formula for Case II should only be considered as nominal. It is unfortunate that the majority of later textbook writers have not repeated this qualification that has caused much misunderstanding of fundamental assumptions. Lack of information about initial basic research has caused many young engineers to assume that the usual stress formula for Case II is mathematically correct and information that this is not the case has been ridiculed.* Strength tests of reinforced concrete beams will ordinarily include the effects of dowel action of stirrups and longitudinal steel reinforcing and allowable nominal shearing stresses with web reinforcement include these effects. It would not appear for this reason to be necessary to make separate evaluations for these influences. One should not assume that those who write codes are unaware of such and other side effects, even though they are not mentioned. It is an accepted rule that code provisions should be brief and simple and only give the most important general regulations. This is not easy to accomplish in many cases and it leads sometimes to uneven safety factors because special conditions cannot be dealt with in a brief document.

The question of dowel action has given raise to a lot of misconceptions. In the first decade of this century some engineers and text book writers considered the shear reinforcing to act similar to rivets in shear where the full steel strength could be developed. A Vierendel bridge was actually built on this principle and the end posts cracked of course. This would not be worth mentioning were it not for the fact that false ideas of contribution of steel reinforcement to shear resistance is still occasionally being propounded. An American textbook on reinforced concrete published in its second edition in 1946 states: "If there is steel passing through the section, the resistance to

*Smith, R. B. L., Discussion of "Shear Diagonal Tension and Anchorage in Beams," ACI JOURNAL, V. 30, No. 12, June 1959 (*Proceedings* V. 55), pp. 1443.

vertical shear is much increased, since the steel must also be sliced off to produce failure," and further: "If there is steel at both top and bottom, there is still greater resistance to shear, and even higher allowable values may be used for the combined action of steel and concrete." Those who have seen examples of failure, would know that the steel does not get "sliced off." The concrete near a crack gets crushed or it splits and the steel bends until it attains a position where tension becomes the principal steel stress. Another false idea which has sometimes been put forth and which was recently seen in connection with a paper on flat plate floors is that the horizontal reinforcing steel could be assumed to add a strengthening effect in shear equivalent to $(n - 1)$ times the steel area. The idea is probably an adaption from reinforced concrete column construction where steel and concrete are stressed in parallel. There is some justification for the assumption in that case but not for the case where a high strength material is supported directly on a weaker one. The weaker material would fail when its compressive strength was exceeded and it would make no difference how great the n value was. The dowel action in reinforced concrete depends chiefly on the contact area, the bending strength of dowels and the concrete strength and has relatively little to do with the ratio of the moduli of elasticity of steel and concrete.

The influence of spacing of bent-up bars is discussed starting on p. 447 and a theoretical explanation is given. This explanation, which the writer has not seen before, appears quite logical. The theoretical maximum distance between points of bending up bars seems also to agree quite well with test results.[62,63] It may not be certain that the same limit will apply fully in such locations where both shear and bending moment is a maximum, because the cracks would then be steeper than 45 deg. In European practice it is considered that the horizontal distance between points of bending up should not exceed d or in some cases even jd or the horizontal projection of the slanting portion.

Referring to pp. 456-459, it was found about 40 years ago in Germany that shear failure could take place even where the nominal shearing stresses were low. It was this finding that caused the requirement in the Germany Code to provide for all shearing stressing along a beam. There has been some doubts if the German tests carried out with relatively low strength concrete would apply also to concrete as used nowadays. The tests by Rodriguez, Bianchini, Viest and Kesler referred to on p. 459 appear to be sufficient proof that the older test results also apply to such concrete as is used in North America. The Canadian National Building Code of 1960, therefore, specifies the use of a minimum web reinforcement along the complete length of a beam even where the shearing stresses are below those usually allowed for concrete without shear reinforcement.

The use of orthogonal web reinforcement is discussed starting on p. 522. There can be no doubt about the efficiency of such reinforcement especially for high beams but horizontal reinforcement in the compression zone is probably of relatively little value. The writer thinks that horizontal reinforcement of this kind is especially useful at points of inflection in beams if there is a possibility of longitudinal forces acting on the beam system.

Beams with bars inclined at angles other than 45 deg are discussed on p. 529. Mörsch states that tests show about as much efficiency for bars bent up at 30 deg as with 45-deg bending.[64] The lesser slope is especially useful for shallow beams and particularly for the innermost bent-up bars. It is even possible, considering the stress trajectories, that bars bent up at 30 deg might be better where there is considerable bending in addition to shear. Mörsch also states that bars bent up at 60 deg are quite efficient and are most suitable for relatively deep and short beams. Within the limits of 30 to 60 deg one may probably choose any convenient angle of bending up bars without much doubt of their effectiveness.

Part 5 of the paper discusses anchorage which is quite an important matter. The provision of an anchorage length of d beyond the point where according to the bending moment the bar can be dispensed with is often insufficient. If a crack forms at 45 deg, that may result in no anchorage at all. Extra anchorage is also required if there is little web reinforcement. These matters were discussed in a paper by the writer.[61]

It is a pleasure to congratulate the authors for the excellent work contained in their paper. The writer would have liked to see some discussion about the effect of concrete shrinkage and plastic deformation on shearing strength. Another problem which is troublesome to the practicing engineer, is the effect of horizontal forces on shear strength. It is hoped that the authors will continue their investigations of the many problems in shear which are still unsolved.

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By R. TAYLOR*

The authors are to be complimented for their comprehensive discussion on the problem of shear in reinforced concrete. In a recent investigation of this problem at the Building Research Station, England, the behavior of beams reinforced with vertical stirrups has been studied. Some aspects of this investigation which are complimentary to, or in conflict with, the findings of the authors are discussed below.

The writer had concluded, as did the authors, that part of the shear force is carried across an inclined cracked section by dowel action of the longitudinal reinforcement. An extreme example of the cranking that can occur in these bars is shown in Fig. M. Owing to the high contact stresses that must occur between the longitudinal bars and stirrups positioned adjacent to a major inclined crack, there will be a considerable frictional restraint to any slip of the main bars. This frictional force will tend to cause some lateral deformation of the stirrups, and an example of such deformation can be seen in Fig. N.

It is interesting to note here that the majority of laboratory tests are, perforce, carried out with steadily applied loads, while actual structures may be subjected to fluctuating loads. In a repeated load test recently carried out at the Building Research Station an unusual type of failure occurred. As can be seen from Fig. O, fracture of a stirrup occurred during the application of a load range in which the maximum was 84 percent of the flexural failing load of similar beams loaded statically. It is considered that fluctuations of the lateral deformation of the stirrup brought about a considerable reduction in its capacity for resisting repeated loads.

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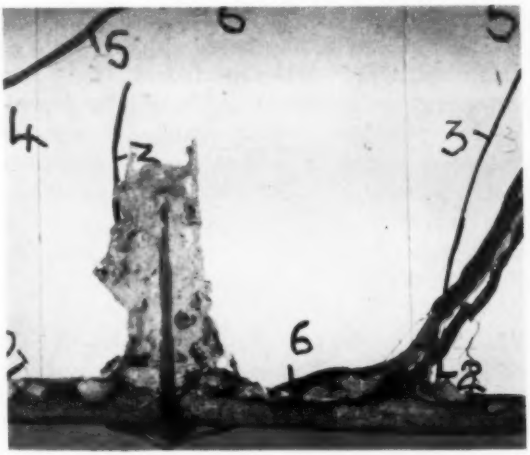

Fig. M — Concrete cover broken away in a beam having widely spaced stirrups to show the cranking of the longitudinal bars between the bottom of an inclined crack and a stirrup

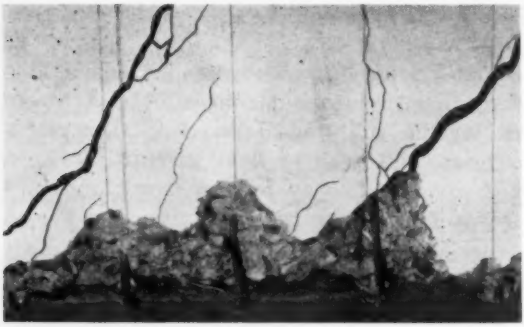

Fig. N — Concrete cover broken away to show the lateral deformation of a stirrup at the bottom of a major inclined crack

Similar reductions due to lateral deformation can occur in certain types of longitudinal bars under the effect of repeated loads. Fig. P shows the fracture of a deformed cold-worked mild steel bar which occurred in a beam within a shear span, i.e., not in the region of maximum bending moment, under a range of loads in which the maximum was 65 percent of the static flexural failing load. It appears likely that the lateral deformation of these bars following inclined cracking caused severe secondary stresses to develop and promoted the brittle fatigue fracture to which cold-worked deformed mild steel bars are more susceptible under the effects of repeated loads than ordinary mild steel reinforcement.

Although the authors have considered lateral deformation in both stirrups and longitudinal reinforcement, they appear to have ignored this effect in bent-up bars. Owing to the size of these bars, it seems probable that considerable load will, in some conditions, be carried across an inclined crack by the dowel action of the bent-up bars. An example of the deformation of a bent-up bar, after failure has occurred along an inclined crack crossing the bar, can be seen in a paper by Bach and Graf.[36]

The authors, from their comparison of Beams 11 and 21 tested by Bach and Graf[32] and reported in Table 9, concluded that the multiple type of stirrup is more efficient than the single stirrup. However, this view is not supported by a similar comparison of Beams 12 and 22 (not reported in the table). It would be of interest to know whether the authors have any experimental evidence for their statement that "multiple stirrups are particularly important in beams with more than three main bars in one layer in a wide web when a single stirrup cannot successfully resist the pressing down of the center bars." In the experience of the writer, the shear-carrying capacity of a beam is reduced when the use of multiple stirrups entails wider spacing.

Attention has been drawn in the paper to the importance of obtaining a satisfactory anchorage of the longitudinal reinforcement when this consists of plain round bars. The writer is of the opinion that similar

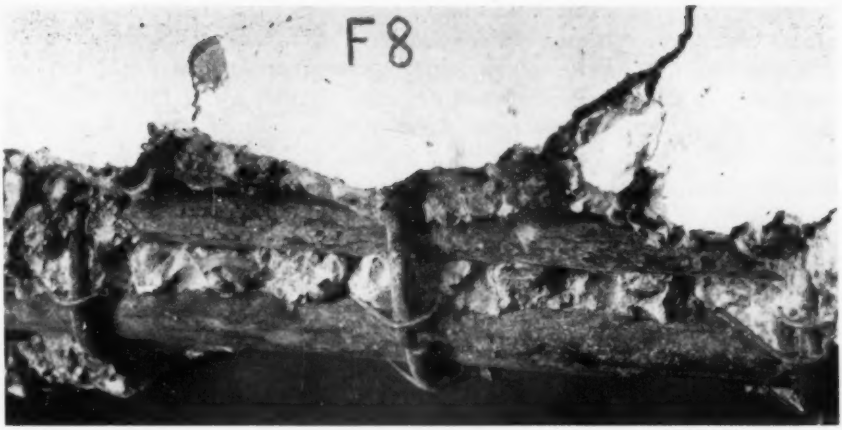

Fig. O—Concrete cover broken away to show the fracture of a mild steel stirrup which occurred during a repeated load test. (Maximum load range was 84 percent of the static failing load)

importance should also have been attached to the anchorage of the stirrups. Since yield of the stirrups generally occurs prior to a shear failure, an adequate anchorage is of primary importance. A study of the types of stirrups used in many of the tests considered by the authors, e.g., those in Reference 32, suggests that anchorage of these stirrups was far from adequate. For instance, this anchorage falls far short of the requirements of the current British Code of Practice,[34] the adequacy of which is also in doubt.[60] In a comparison of beams having the type of hooked stirrup shown in Table 9 with beams having stirrups continuing straight to the top surface, Bach and Graf[32] found no difference in the average failing loads, although, not surprisingly, considerable slip of the ends of the 'straight' stirrups was observed. The writer would conclude from these tests that the anchorage of the

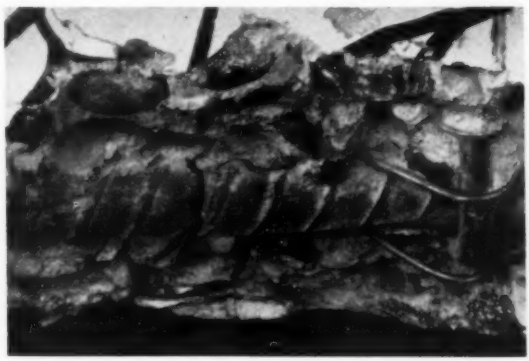

Fig. P—Concrete cover broken away to show the fracture of a deformed cold-worked mild steel bar that occurred within a shear span during a repeated load test (maximum load range was 65 percent of the static flexural failing load)

hooked stirrups in Table 9 was also unsatisfactory. This is an important point, since the authors have concluded from the tests reported in Table 9 that the proportioning of stirrups in accordance with the British Code does not offer a complete safeguard against shear failure. In the writer's view, the provision of adequately anchored stirrups proportioned in accordance with the British Code will fail to provide security against shear failure only when there is a tendency for the concrete in the web of the beam to crush under the action of the inclined compressive stresses. This type of shear failure has been discussed elsewhere.*

By W. WELEFF†

The importance of the shear stresses on the stability and carrying capacity of reinforced concrete structures has been recognized almost from the infant days of reinforced concrete. The Hennibique's patents of 1892 are considered an improvement of the first Monier's inventions of 1867 and these improvements were made providing bent-up bars for taking care of the shearing stresses. Ever since the shear resistance of plain, reinforced, and most recently of prestressed concrete has been investigated many times and some consideration for adequate shear reinforcement made.

In investigating, however, the causes of failures in reinforced concrete structures, a considerable amount of these failures is attributed to inadequate shear reinforcement. This fact is showing that despite the importance of the shearing stresses in the stability and safety of the structure, inadequate care or insufficient understanding of the problem (or in some cases ignorance) are the contributing factors for the lack of adequate provisions.

From this point of view, the excellent contribution of the authors will bring a better understanding of the shear problems involved in the design of reinforced concrete beams. Unfortunately the authors are limiting their presentation to the shear problems of reinforced concrete beams only. For completeness it is felt desirable to include all reinforced concrete structures such as shells, dams, arches, etc. Where stresses in a structural member are involved there is most probably a shearing stress also present which requires considerable attention.

The main objective of this discussion here is threefold:

1. To point out some additional areas in reinforced concrete beams where the shear stresses and provisions for adequate shear reinforcement are to be considered.

*Taylor, R., Discussion of "Some factors in the shear strength of reinforced concrete beams," by A. M. Neville and E. Lord, to be published in *The Structural Engineer* (London).

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2. To point out the inadequacy in the ACI Building Code pertaining specifically to shear reinforcement

3. To stress the necessity for a revision of the pertinent chapters of the ACI Building Code requirements dealing with shear reinforcement

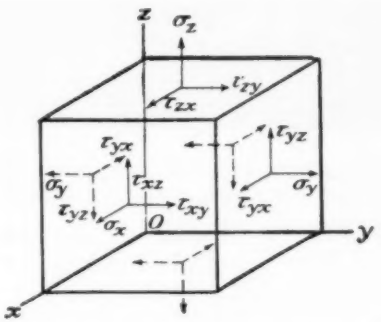

Fig. Q—Three dimensional stress distribution

With the increasing tendency of design engineers and architects to use large areas of reinforced concrete structures as carrying members such as shells, folded plates etc., the two and three dimensional stress distribution is bound to get more attention. It is readily seen that in a structural member generally carrying stresses in two or three directions, not only the shearing or tensile stresses in the xy-plane are to be considered, but also the stresses in the other two planes, viz., the xz- and yz-planes (see Fig. Q).

For example, the shearing stress at the end of a beam in the lateral direction is often neglected by most designers. Fig. R shows cracks at the end of a reinforced concrete beam indicating insufficient reinforcement in the direction of the width of the beam. In the longitudinal direction bent-up bars and stirrups are provided to take the full shearing stress. This problem is well recognized by the prestressed concrete designer, who is forced more or less due to the magnitude of the force in the tendons to provide additional longitudinal as well as vertical and transversal end-block reinforcement, or certain types of spiral reinforcement. The prestressed concrete engineer encountered these difficulties early in the development stage. This problem has also been recognized by some European authorities.

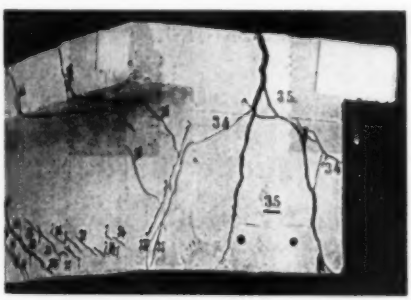

Fig. R—Shear cracks at end of reinforced concrete beam

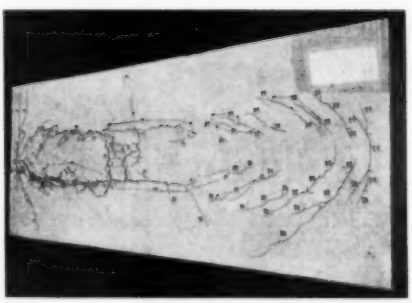

Fig. S—Surface hair cracks above edge of beam

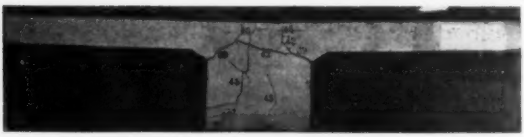

Fig. T—Shearing stresses at beam-slab junction

An additional area, where the shearing stresses of reinforced concrete structures deserve more consideration, is the surface area of concrete slabs above the edges of the beams. It is almost traditional to accept hair cracks at these areas as shown in Fig. S. These shear cracks can be prevented, if so desired, by providing the necessary top reinforcement. It should be pointed out here that as long as these hair cracks are in the compression zone, no imminent danger for the structure is to be expected. However, the concrete area carrying the compressive stresses may be reduced considerably, depending on the amount of these cracks. Above interior supports of continuous beams or frames, the situation is even worse. If such an area is exposed as floor space, where mechanical damages are possible, or acids and other chemicals are used, this will weaken the structure rapidly.

Finally, an area where the shearing stresses are often neglected is the junction of the slab and beam. Fig. T shows the shear cracks in this area. Once longitudinal shear cracks occur the slab and the beam are no longer a homogeneous section and it is not difficult to predict what will happen by overload.

This discussion should be considered mainly in connection with the ACI Building Code. The inadequacy of its provisions on shear have been recognized for a long time in the engineering profession as well as in the reinforced concrete industry, as indicated by the large discussions following the latest edition of the ACI Building Code and also by the numerous articles appearing since then in the JOURNAL and the other civil engineering literature.

The many valuable suggestions of the authors' presented in their series of articles should be incorporated in a future revision of the Building Code. It is inappropriate here to repeat them. The following remarks are to be considered supplementary to the numerous recommendations made by the authors:

1. The allowable shearing stress for plane concrete is considered to be too high and should be lowered.

2. No distinction should be made between reinforced and plane concrete for the allowable shearing stress.

3. Wherever the shearing unit stress exceeds the allowable one, shearing reinforcement shall be provided to carry the full shearing stress over the total length of the beam.

4. The total shearing stress for non-homogeneous members should be carried by shear reinforcement over the entire length.

5. The web reinforcement shall consist of stirrups and bent-up bars.
6. The bent-up bars as well as the stirrups shall be anchored in the compression zone.
7. Appropriate investigation of the shearing stresses in all directions of the carrying structural member are to be included.

AUTHORS' CLOSURE

The numerous and thoughtful contributions to the discussion on this series of papers on shear are most gratifying, but the volume of material presented makes it possible to comment only on some of the points raised.

Professor Ferguson is obviously right in saying that cross beams have a stiffening effect on the main beams. For instance, as shown by the authors,[1] the cross beams reduce the deflection compared with a prismatic beam, when both types are loaded directly. This effect is, however, not necessarily related to the method of loading since, as shown in the paper, beams loaded unsymmetrically are equally likely to fail at either end, so that the mode of loading has no decisive influence on the location of the diagonal tension crack. The size of the cross beam is also not thought to be of great importance, and in fact the cross beam at the support was proportioned realistically and is by no means massive. In any case, the strength of the cross beam is quite distinct from the "good connection" believed by the authors to be necessary. Indeed, the authors investigated the type of beams encountered in practice and thus ones with connections sufficiently strong not to encourage premature failure; there were no stirrups in the main beam (see Fig. 13). There is no doubt, however, that Professor Ferguson's remarks on failure "near and not across" the support are correct as the vertical compression stresses do act at the support, and this the authors never disputed.

The authors would not consider Taylor's[2] results (referred to by Professor Ferguson) as conclusive since in that investigation no duplicate tests were made, so that it is hard to distinguish between the scatter of results and real differences.

Fig. M and N of Mr. Taylor's are interesting and support some of the authors' suggestions on the action of longitudinal reinforcement and stirrups in beams failing in shear. His data on beams subjected to repeated loading are particularly valuable, and it is hoped that he will publish more complete information on these tests before long.

Mr. Taylor seems to have misunderstood the recommendations on the use of multiple stirrups. The authors should explain that they certainly did not advocate a wider spacing of stirrups: indeed, suggestions on maximum spacing were made in the paper but the authors believe that for a given spacing it is desirable to use multiple stirrups

of correspondingly smaller diameter than single stirrups when there are three or four bars in one layer in a wide web.

The authors are in agreement with Dr. Hajnal-Konyi's and Mr. Taylor's remarks on the anchorage of stirrups. However, they do no think that a satisfactory anchorage of stirrups is a complete safeguard against shear failure; numerous tests, including some made by the authors and quoted in the paper, have shown that shear failure can nevertheless occur because satisfactory anchorage of stirrups does not prevent their yielding.

The authors are pleased that an engineer of Dr. Abeles' standing agrees with their suggestions on the basis of design for shear and on the spacing of stirrups. They differ, however, on the question of prevention of shear failure: although satisfactory resistance to shear can be obtained by the use of adequately spaced vertical stirrups this type of reinforcement may not altogether prevent shear failure. Nevertheless, it can be argued that in many cases the mode of ultimate collapse is of lesser importance than a high enough shear-resistance of the beam. The question of the choice of inclined or vertical web reinforcement depends on economic factors, not the least of which is the cost of labor, and the authors did feel that any generally applicable statement could be made.

Professor Posey's contribution on the subject of anchorage is interesting. Unfortunately, in many beams the use of hooks with a hook-to bar-diameter ratio, k, of 11 is not possible because of a lack of space, and under such conditions the remedial measures suggested by the authors may increase the load capacity of the beam.

Professor Ferguson, Dr. Abeles and Dr. Hajnal-Konyi give various examples of bond failure. The authors certainly did not intend to imply that bond is not critical in splices or in beams where shear is sufficiently well resisted. They ignored this type of failure, as being outside the scope of the paper, and regret that they did not sufficiently qualify their statement on the secondary nature of bond failure following the development of diagonal tension cracks and general distress in shear. The words "*normal* beams subjected to both flexure and shear" at the bottom of p. 729 and the top of p. 730 should have been in italics. This qualification would, for instance, exclude Walther's beams with polished bars, referred to by Dr. Hajnal-Konyi. The authors feel, however, that the secondary character of the loss of bond in a beam failing in shear is of importance, as in such a beam shear failure would occur even if the design bond stress was quite low. In this connection Dr. Hajnal-Konyi's statement that "the resistance of bars to bond does not depend on the bond stress alone but also on the shearing stress of the concrete" is of interest.

The calculation of permissible bond stress in beams with bent-up bars prescribed by the German code and quoted by Dr. Hajnal-Konyi is not based on rational analysis, but recognizes the influence of shear reinforcement on the apparent strength in bond. Furthermore, no reduction in the shearing force is given in the British or ACI codes, which formed the basis of the authors' calculations.

Mr. Rensaa's contribution was read by the authors with great interest. The authors should explain, however, that their data on T-beams were not meant to apply to inverted T-beams and no suggestion was made that inverted T-beams can sustain a higher shear than rectangular beams. Transverse stresses in longitudinal and web reinforcement may not be critical but they should be included in a study of the behavior of beams under the action of shear and flexure. Ignoring real factors and simply using allowable stresses is not justified at this stage and, in Mr. Rensaa's own words, may set a "trap for those who deal with unusual cases." Mr. Rensaa may be interested to hear that the lack of influence of compression reinforcement on the shear strength of a beam was confirmed experimentally and reported in an earlier paper.[3]

Mr. Weleff's valuable recommendations should be endorsed; they show once again that in the past insufficient attention has been paid to shear.

Mr. Erdei has obviously done a tremendous amount of work on the subject and it is not possible to discuss his work fully in this closure. His arguments are generally in agreement with those of the authors, e.g., on the influence of the diagonal tension cracks on the distribution of bending moment on the beam. The authors are not clear, however, why it is always considered to be smaller than $d(1-k)$: indeed in Mr. Erdei's Fig. E. the two quantities appear equal. Regarding his comments on Beam 38 (Fig. 21) the authors feel that it is precisely because of the heavy web reinforcement that cracks open in a nearly vertical direction as in that direction there is less resistance to the widening of the crack; in other words, bending stresses are less well resisted than those arising from shear. Mr. Erdei quotes other examples of steep cracks but these always open when flexure has a predominant influence on failure, which occurs under a load approaching the moment of resistance of the beam. Such failure is, nevertheless, not independent of the shearing forces acting, as for instance is the case with Morrow and Viest's beams considered by Mr. Erdei. While the authors have no first hand knowledge of these beams it appears that they did indeed fail in shear and this cannot be disproved by the code formulas, quoted by Mr. Erdei: such formulas are only estimates and generally conservative ones. Likewise, the value of τ_{max}/f_c' given by Mr. Erdei is no more than typical, as the ratio of the tensile and compressive strengths of concrete varies between over 0.15 and approximately 0.07 depending on the level of strength.

Mr. Erdei's remark that there is no permissible shear stress below which shear need not be considered is interesting as it focuses our attention on the necessity for careful design of members subjected to shear.

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62. Taylor, R., "Some shear tests in reinforced concrete beams without sheer reinforcement," *Magazine of Concrete Research* (London), V. 12, No. 36, Nov. 1960, pp. 145-154.

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ACI NEWS LETTER

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memo from the President:

Hot Weather Concrete Operations - A Reminder

James Russell Lowell said, "And what is so rare as a day in June? Then if ever come perfect days."

We agree with him that warm and sunny days are perfect for many things, but not for concrete placing operations. The perfect days for concrete work were the cool, damp days of spring. Usually the summer days become less favorable as the season advances into warmer and drier weather.

Where hot weather has not already arrived, it may be expected shortly by most of our members. Accordingly, it seems worthwhile to recall, and recommend for rereading, the report of Committee 605 which now is the ACI standard "Recommended Practice for Hot Weather Concreting (ACI 605-59)." This was published in the JOURNAL in November, 1958, p. 525. Reprints are available.

Concern for concrete placed in hot weather goes beyond trouble with finishing, greater slump loss, early stiffening, and cold joints. High temperature, like most sources of acceleration of hardening and strength development, impairs ultimate strength. There are few mixes which do not show lower 28-day strengths when mixed and placed at appreciably higher than usual temperatures. Mixing water requirement is increased and drying shrinkage is increased proportionately. Attention to curing becomes more important and it requires more ef-

fort to get it. Moreover, concrete that hardens at a high temperature has a considerably greater thermal shrinkage when it cools to the low temperature of next winter, than concrete placed under controlled lower temperature conditions.

These are all good reasons why it will usually be worthwhile to do at least some of the things which can be done to offset such results. The temperature of concrete as placed can be kept down to 90 F or less by such relatively inexpensive means as sprinkling and shading aggregate supplies, cold mixing water, and perhaps some flake ice directly in the mixer.

Such nominal control of maximum temperature, however, will ordinarily be insufficient to prevent the usual hot weather difficulties in handling, placing, and finishing, or to offset the usual increase in water requirement and drop in strength. These conditions can be helped materially by proper use of one of the several reliable water reducing and set retarding admixtures now available. The increase in thermal shrinkage mentioned can be avoided only by reducing the concrete temperature to a nominal value. Curing will always require special care and attention.

Hot weather concreting is a special problem and both immediate and ultimate results will notably respond for the better if the well-proved practices outlined in the standard ACI 605-59 are followed.

Lewis H. Tuthill
President

Two-story office building completed in less than a month

A modern two-story office building ready for occupancy just 28 days from the placing of the first column was the building record set by Snapp Construction Co. in the erection of the firm's new quarters in Leesburg, Fla.

Precast prestressed framing members and concrete double tees were used. The building frame was erected in just 28 working hours. On the first day, footings were completed and actual construction began by placing of columns. At noon of the second day (left) the first double-tee roof section was placed on the supporting beams. The prestressed concrete double tees were cantilevered to give wide eaves, generous balconies, and an extra office space suspended over the driveway.

Institute Goals Updated

At the Feb. 19, 1961, meeting of the Board of Direction an important report was made by the special Board Committee on Aims and Objectives, W. J. McCoy, chairman. Other members of the committee are: Douglas McHenry, Walter H. Price, Paul F. Rice, C. H. Scholer, J. Neils Thompson, and Stanton Walker. The committee by correspondence and a series of meetings reviewed the Institute goals adopted in 1948, evaluated progress toward their accomplishment, and studied new areas of likely ACI activity. As a result of the efforts of this group the Board of Direction adopted the following as a statement of Institute policy as reasonable objectives for ACI efforts:

1. Maintain constant surveillance to assure that papers cover primary member interests with particular emphasis on effective and concise presentation. Inclusion of more papers describing methods of construction, architectural features, and details of techniques successful in obtaining attractive and durable structures as an immediate objective. Publication of the monograph series should be encouraged and expedited.

2. Increase in quantity and scope of technical committee reports, thereby increasing the usefulness of such reports.

3. Maintenance of the high technical value of conventions and regional meeting sessions with emphasis on current interests and improving techniques of presentation.

4. Increase in number and significance of discussion of committee reports and technical papers. It is important that a progressive attitude be maintained with respect to ACI publications and at the same time to retain their authoritative character. It is also important that all segments of the Institute's membership have an opportunity to express their views.

5. Continued emphasis on taking the Institute to the members by increasing the number and activity of local chapters and by increasing the number of locally sponsored meetings and conferences.

6. Increase in efforts through planned public relations activity to advise nonmembers of the useful technical information and the opportunity for active participation that is available from ACI.

7. Accelerated increase in membership to ultimately include substantially all persons who can be usefully served by the Institute.

8. Increase in opportunities for members outside the United States to participate in the work of the Institute.

9. Periodic re-evaluation of the administrative structure of the Institute to maintain a proper balance of responsibilities among Board, Technical Activities Committee, Standards Committee, technical committees, and staff.

10. Encouragement of continuing service by those taking an effective part in technical committee work and close review of committee performance in order to maintain active committees.

11. Increase in effort to stimulate research aimed at enlarging basic knowledge of concrete or improving its properties as a construction material.

County bridges in prestressed concrete

County engineers are finding that prestressed concrete is a natural for the replacement of outmoded bridges and new structures.

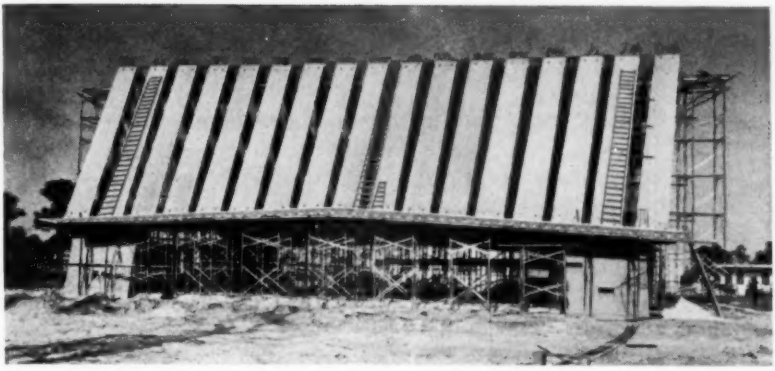
An example of an attractive prestressed county bridge is the one for Franklin County, Pa., near Chambersburg. Franklin County took alternate bids for prestressed box beams, prestressed composite I-beam construction, and composite steel construction. In this case the box beam design proved most economical and the total bridge was built for $10.73 per sq ft. The consulting engineers were Frederic A. Nassaux & Associates, Chambersburg, Pa. New Enterprise Stone & Lime Co., Roaring Springs, Pa., supplied the prestressed box beams.

Harvey County, Kan., also built a prestessed box beam bridge. The bridge, over Sand Creek, is 255 ft long with two outside spans of 49 ft and three interior spans of 51 ft. The bridge is of composite design. Mild steel reinforcing was placed over the supports in the concrete topping to make the bridge continuous for live load. The box beams rest on Neoprene bearing pads. The piers are prestressed concrete pile bents. The completed bridge was constructed for $8.08 per sq ft. The structure was completed in 36 working days. The bridge was designed for Harvey County by Wichita consulting engineer R. S. Delamater.

Sever other counties (Linn County, Ore.; Whitman County, Wash.; Lincoln County, Wash.; and Racine County, Wis.) have had good success with prestressed single T-section. One such bridge across the Amazon Creek in Eugene, Ore., with a 70-ft span cost only $6.73 per sq ft. These bridges are usually designed with a composite concrete topping and supported on synthetic rubber bearing pads.

Prestressed bridge for Franklin County near Chambersburg, Pa.

Double tee beams are 45′ long, 4′ wide, 15″ deep, and weigh 5 tons. The side beams, which support the double tees, are 95′ long, 9′ wide, 12″ thick, have a 2′ rise at the center, and an exposed aggregate finish.



PRESTRESSED DOUBLE TEES

for unique roof structure . . .

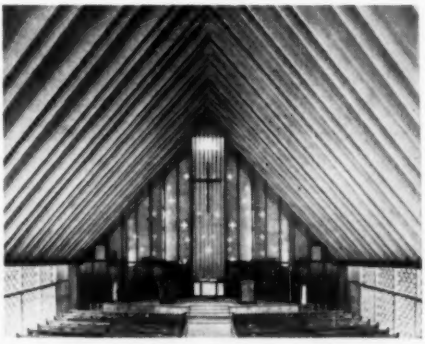
● The design of Melrose Park Community Methodist Church, Ft. Lauderdale, Fla., called for a new and imaginative use of standard prestressed concrete double tees. Capped with concrete at the top and secured at the bottom on two huge cast-in-place beams, the 34 double tees make up the "backbone" of the roof structure. The 24″ open spaces between the double tees were filled with a pneumatically placed mortar. The end result is, in effect, a single, giant folded plate supported only by the corner abutments.

In the manufacture of the double tees, R. H. Wright, Inc. used Lehigh Early Strength Cement. Consistent use of this type of cement in their prestressing operations helps them attain maximum production efficiency through early removal of units and quick re-use of forms.

This is typical of the advantages of Lehigh Early Strength Cement in modern concrete construction. Lehigh Portland Cement Company, Allentown, Pa.

LEHIGH CEMENTS

Non-load-bearing precast columns support 3′ x 8′ terrazzo facade panels. Bell tower is 85′ high, 6′ wide at the base, and was precast in 9 sections with Lehigh Early Strength Cement. Abutment structures are 10′4″ x 5′ with 12″ walls, and have an exposed aggregate finish.



Acoustical plaster was sprayed over exposed double tees. The structure presently seats 510 with future plans calling for the addition of wings to increase seating capacity to over 1,000.

Architect: Robert E. Hansen, A.I.A.
Engineer: Walter Harry Assoc.
Contractor: Casmal Construction Co.
Manufacture and Erection Prestressed/Precast Concrete: R. H. Wright, Inc.

All of Ft. Lauderdale, Florida

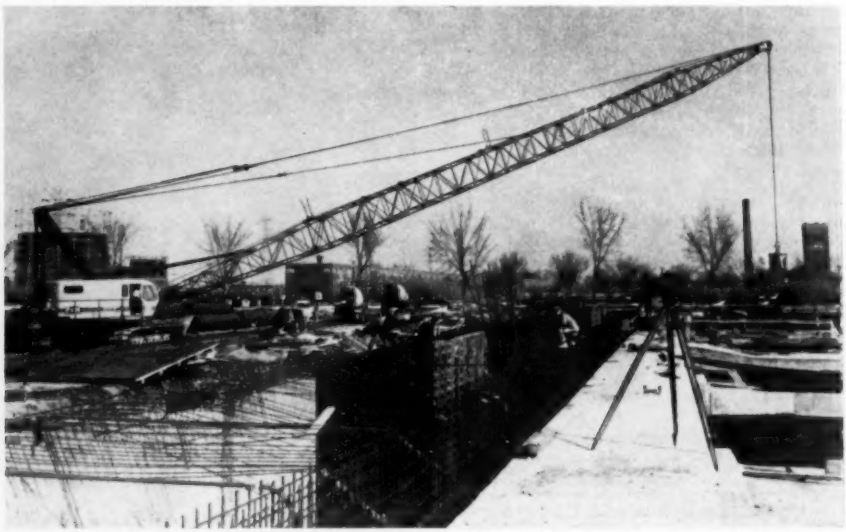
A 'Long Reach' was Needed For This Job

A crawler erecting crane with a long "reach" proved to be a key tool in construction of six aeration tanks and eight circular tanks at the North Side Treatment Works of the Metropolitan Sanitary District of Greater Chicago.

In figuring the job, J. L. Simmons Co. of Chicago recognized that the exceptionally compact layout for the project would call for working from the perimeter if the job was to be completed economically. This was particularly true because engineering called for the six aeration tanks to be joined to each other along their 290-ft lengths by common inner walls, forming a rectangle 410 ft long and 290 ft wide.

By utilizing a 110-ton P&H Model 1015 crawler crane with a 200-ft boom and a 1-cu-yd bucket it was possible to reach out more than 170 ft to place concrete. This way all concrete could be placed from outside the rectangle's perimeter, thus avoiding expensive delays in the building of forms and the placement of reinforcing steel.

Simmons personnel feel they were successful in planning maximum daily production into their handling of the project. In meeting the tough requirements of placing concrete for the center areas of the aeration tanks (90 to 170 ft out from the machine), the contractor equipped the 200-ft boom with a 1-cu-yd bucket. Then, to achieve maximum speed in placing the outer



Crane with a 200-ft boom swings a 1-cu yd bucket of concrete into place during concreting of the aeration tanks of sewage treatment facilities for Chicago

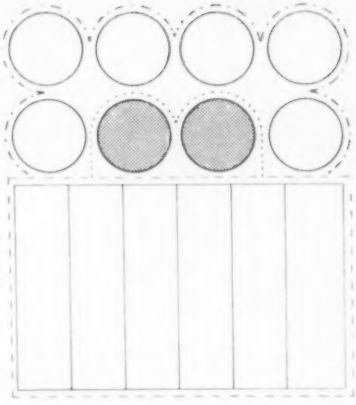
walls, this bucket was replaced with a 2½-cu yd bucket.

Another advantage to working from the perimeter at all times was the relatively easy delivery of concrete directly from the mixer trucks to the crane bucket.

Besides placing concrete, the crane was also used in moving forms and steel from the perimeter to points within the six-section rectangle. A smaller 25-ton truck mounted crane was used to move materials around the perimeter and to spot loads for the larger machine.

Construction of the eight circular settling tanks followed a similar pattern. Work began on the two center tanks in the first row directly north of the aeration facilities, with the other six circular tanks built around the initial two. Thus the end product of the settling tank area was two rows of four tanks each, forming a second rectangle 410 ft long and 240 ft wide.

The long reaching crane was also used to spot heavy activating mechanisms in each settling tank after the concreting was completed. Still another problem was the fact that the settling tanks were located only 2 ft apart, but the ground level between them had to be raised after the outer wall forms were removed. With the long boom, dirt loads were spotted and dumped between the tanks.



J. L. Simmons Co., taking advantage of the 170-ft reach of the boom on the crawler crane, first worked around 290 x 410-ft perimeter of aeration tanks (dashes). Then, two settling tanks (shaded) were built with crane moving along dotted path. Finally, working from sides of settling tank area (dash-dot path) the last six circular tanks were concreted

Sozen among '61 Epstein award winners

Mete A. Sozen, associate professor, was one of three members of the University of Illinois Department of Civil Engineering faculty, selected as the 1961 winners of the A. Epstein Memorial Award for Faculty Achievement.

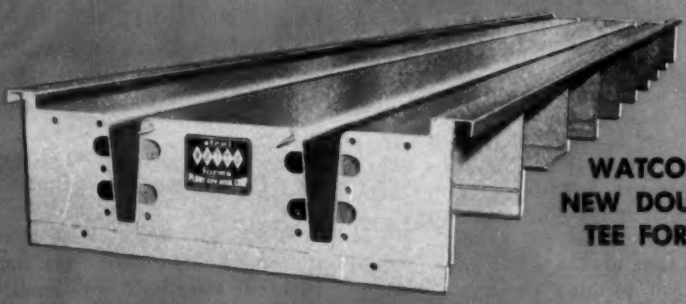
Professor Sozen, one of the staff since 1953, graduated from Robert College, and holds MS and PhD degrees from the University of Illinois.

Ben B. Ewing, associate professor of sanitary engineering, and Houssam M. Karara, assistant professor of civil engineering, were the other award winners.

Selection was based on the achievement of the candidates, their scholarship, professional standing, distinction in public service and in service to the department, and on their general overall contribution to the prestige of the Civil Engineering Department and of the University of Illinois.

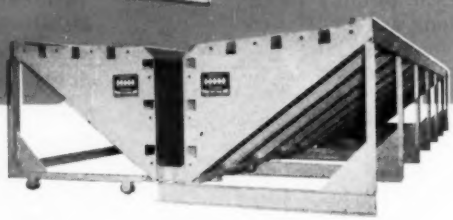
Funds for the A. Epstein Memorial Award were contributed by the firm of A. Epstein and Sons, Inc., Chicago, and particularly Raymond and Sidney Epstein, graduates of the Department in 1938 and 1943. From 1955 to 1958, A. Epstein, a UI CE graduate in 1911, provided funds for a faculty achievement award. Following his death in 1958 the sons continued the award.

What's New from watco



WATCO'S NEW DOUBLE TEE FORM

WATCO'S NEW SINGLE TEE FORM



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Positions and Projects

ACI Oklahoma Chapter officers for '61

John Kemp, General Portland Cement Co., Oklahoma City, heads up the Oklahoma chapter as president for '61. Stanley Gerlach, Gerlach Co., is vice-president, and James Sprouse, The Master Builders Co., is secretary-treasurer; both are from Oklahoma City.

John Kemp

Past-President R. H. Jackson becomes a member of the board of directors of the Oklahoma Chapter. Other board members for the current year include L. W. Baxter, Portland Cement Association, Tulsa; Glenn Wheeler of Wheeler and Wheeler, Enid; and H. B. Wenzel, Dolese Co., Oklahoma City. Retiring directors are Ray Kraft, Refinery Engineering Co., Tulsa, and Dan Hapke, PCA, Oklahoma City.

The 1961 officers, previously elected by ballot, were installed by Robert P. Witt, professor, Oklahoma State University, Stillwater, at the third annual meeting of the chapter on March 15.

Professor Witt, out-going chapter vice-president, acted as master of ceremonies for the dinner-meeting. Guest speaker of the evening was A. Allan Bates, member of the ACI board of direction, president of the American Society for Testing Materials, and a member of the executive committee of the Building Research Advisory Board of the National Academy of Sciences. Dr. Bates narrated pictorial slides obtained while on a tour of European countries. William A. Maples, ACI secretary-treasurer, attended the meeting, speaking on pending publication of manuals and construction data for use by engineers and architects.

Baldwin joins JOURNAL staff

Robert C. Baldwin joined the ACI JOURNAL staff as an assistant editor on April 25. Mr. Baldwin, a recent graduate of Kansas State University, Manhattan, received dual degrees in civil engineering and technical journalism. While at Kansas State he served on the staffs of the daily *Kansas State Collegian* and the *Kansas State Engineer*. He holds an EIT certificate in Kansas and is affiliated with NSPE and ASCE.

ACI technical committee chairman reappointments

Acceptance has been received by the following committee chairmen who have been reappointed for 1 year because of work in progress.

Committee 334, Concrete Shell Structures
Anton Tedesko
Roberts and Schaefer Co.
New York, N.Y.

Committee 401, Specifications for Structural Concrete
George H. Nelson
Law Engineering Testing Co.
Atlanta, Ga.

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 209, Creep and Volume Changes in Concrete
John R. Keeton
U.S. Naval Civil Engineering Lab.
Port Hueneme, Calif.

J. T. McCall
Lock Joint Pipe Co.
East Orange, N.J.

A. M. Neville
Nigerian College of Technology
Zaria, Nigeria

Committee 334, Concrete Shell Structures
David P. Billington
Princeton University
School of Engineering
Princeton, N.J.

Committee 341, Reinforced Concrete Columns
Phil M. Ferguson
University of Texas
Austin, Tex.
Sidney A. Guralnick
Portland Cement Association
Skokie, Ill.
Ivan M. Viest
AASHO Road Test
Ottawa, Ill.

Committee 334, Concrete Shell Structures
David P. Billington
Princeton University
School of Engineering
Princeton, N.J.

Committee 403, Adhesives for Concrete
M. R. Smith
Department of the Army
Office of the Chief of Engineers
Washington, D.C.

Committee 612, Recommended Practice for Curing Concrete
Charles E. Proudley
Carolinas Ready Mixed Concrete Association
Raleigh, N.C.
V. C. Saville
Missouri State Highway Commission
Jefferson City, Mo.

Committee 717, Low Pressure Steam Curing
J. T. McCall
Lock Joint Pipe Co.
East Orange, N.J.
Lee Pohsner
Consolidated Engineers Associates, Inc.
Boston, Mass.

Committee 805, Application of Mortar by Pneumatic Pressure
John C. Fredericks
Gunite Contractors Association
Los Angeles, Calif.
J. T. McCall
Lock Joint Pipe Co.
East Orange, N.J.

Re-Steel Discussed at Michigan Chapter Meeting

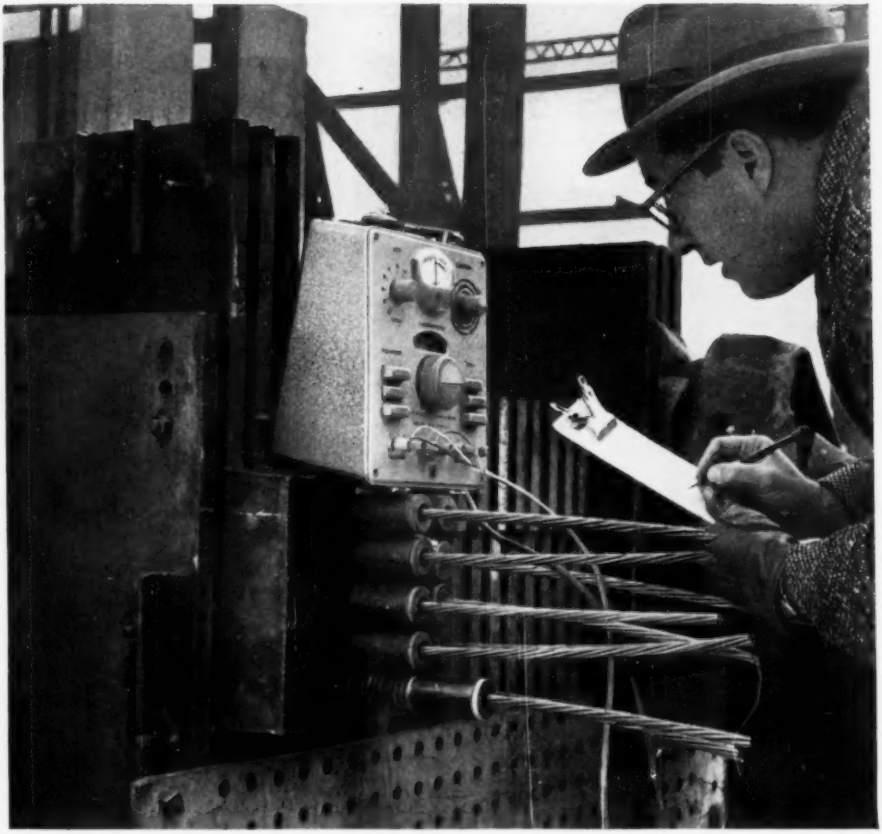
The April meeting of the Michigan Chapter of the American Concrete Institute, held in Detroit, featured a discussion on high-strength, fine-grained, carbon reinforcing steel. C. L. Altenburger, technical assistant to the president in charge of research and development, Great Lakes Steel Corp., Ecorse, Mich., was the speaker. He discussed columbium-treated, mild carbon, semi-killed reinforcing steel.

Mr. Altenburger described the composition of the steel. He noted that the high-strength steels discussed consisted of fine-grained, semi-killed, mild carbon, steels treated with varying amounts of columbium. Columbium refines the grain structure and this in turn means that the tensile properties of the steel are increased to high values and ductility is little changed. He described chemical composition and mechanical properties. He noted that the reinforcing bars were weldable without special precautions as to technique and electrodes; that the bars can be bent almost flat on itself; and are available in yield strengths of 45,000, 50,000, 55,000, and 60,000 psi.

Following the custom originated by the Michigan Chapter, William C. Krell, chapter president, presented an ACI lapel pin to Alfred Zweig, senior structural engineer, Albert Kahn Associated Architectural Engineers, Inc., in recognition of his being a speaker at one of the monthly luncheons during the year.

Freseman joins ACPA staff

Ray C. Freseman was recently appointed assistant to the managing director of the American Concrete Pipe Association, Chicago. He was previously with Meissner Engineers, Inc., Chicago, as a design engineer specializing in foundation design.



How Bethlehem Engineers Aid Prestressed Concrete Fabricators

Friction-loss test conducted to meet Pennsylvania Highway Department specs

Meet Jim Scott, a Bethlehem research engineer. He is recording the micro-inch differential of Bethlehem stress-relieved strand under load at the Dunmore, Pa. plant of Concrete Engineering, Inc.

Jim is there because of a stringent test requirement of the Pennsylvania State Highway Department. To qualify as a supplier of deflected-strand prestressed members for highway bridges, a fabricator must furnish evidence that the frictional loss in his strand, from one abutment of his casting bed to the other, is no greater than 5 per cent. A measurement within this tolerance is accepted as proof that the prestresser's method of operation is satisfactory.

Under the watchful eyes of State Highway Department witnesses, Bethlehem conducted a 4-station load cell test. Tension links were used on 2 slopes and compression links at both ends of the 345-ft bed. Test results showed frictional losses of less than 3 per cent, well within the state requirements.

This sort of assistance is an example of how Bethlehem is prepared to help *you*. Our service does not end with the delivery of top-quality stress-relieved strand. Bethlehem also offers field engineering service, whenever needed. If you need technical assistance, simply call the nearest Bethlehem sales office.

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BETHLEHEM STEEL

Prestressed circular reservoirs will supplement existing water storage facilities at Springfield, Mass.

Prestressed Water Reservoirs

Two large prestressed concrete tanks are now being constructed to supplement existing water storage facilities at Springfield, Mass. The cylindrical tanks measure 320 ft in diameter and are 28 ft high. Each tank will hold 15,500,000 gal. of water. Floor slabs are 5 in. thick, walls are 18 in. thick, and the column-supported roofs will be 9 in. thick.

The prestressed concrete walls are wound with some 400 miles of prestressing wires to withstand the lateral forces which will be exerted by the water when the tanks are in service.

Prestressing comprises more than 1000 wraps of wire applied by a self-propelled wire-winding machine which was suspended along the sides of the tanks from a carriage that rode around the tops of the walls. The wire was drawn from a diameter of 0.236 in. to 0.208 in. as it was wrapped, imparting an initial tension in the wire of 150,000 psi. The inward forces thus applied to the tanks range from 336 kips per vertical ft at the bases, which are prestressed with four layers of wire, to 28 kips per vertical ft at the tops, which have only one layer of wire. On completion of the prestressing operations, the wires were covered with shotcrete. The tanks ultimately will be backfilled and covered with a 2 ft thickness of earth.

The economies effected by prestressed construction are readily apparent when the 18 in. wall thickness of the new tanks is compared with the tapered walls of the old rectangular reservoir shown to the left of the new tanks in the photo. These walls range from over 3 ft thick at the base to some 2½ ft thick at the top, approximately 1⅔ to 2 times as thick, with construction costs to match.

The owner of the new tanks is the city of Springfield, Mass. Clinton Bogert Engineers, New York, are the consultants. General construction is a joint venture by Fruin-Colnon Contracting Co., Indianapolis, Ind., and C & C Construction Co., Ft. Wayne, Ind. Prestressing was by The Preload Co., New York.

Electrovert appoints Kolozs to vice-presidency

John E. P. Kolozs has been appointed vice-president of marketing for all construction, electrical components and electrical and electronic equipment for Electrovert Inc., New York. Mr. Kolozs was previously in charge of the construction materials division.

ACI grant to Reinforced Concrete Research Council

Recognizing the critical need for increased financial support to research in the reinforced concrete field the Institute Board of Direction recently approved a grant of $2000 to the Reinforced Concrete Research Council, the technical and financial coordinator in the United States for support of reinforced concrete research at universities. Further financial support will be considered in the next year's ACI budget.

The RCRC is a forum for research in reinforced concrete, and its work is carried on jointly by outstanding men in science, education, government, and industry. The council was organized in 1948 with all funds contributed being applied wholly to support and maintain a vigorous program of research at engineering laboratories of various universities.

The policy of the council is to foster, sponsor, finance, supervise, and in every way possible support research aimed at solving reinforced concrete problems. Preference is given to projects proposed by technical committees of professional societies which are encouraged to submit to the council specific requests for research related to analysis, design, and construction. Thorough consideration is also given to projects within the council's objectives which are proposed by others. Special attention is given to projects submitted by engineering colleges for council sponsorship.

In the Reinforced Concrete Research Council professional societies, industry, government agencies, engineering organizations, and universities join together to advance scientific knowledge of reinforced concrete through research. Although several projects are currently being financed by the council there is need for additional funds to support the backlog of important investigations.

8th annual concrete conference at U of ND

The 8th annual concrete conference at the University of North Dakota, Grand Forks, was held April 10-11.

Approximately 125 individuals attended the conference hearing a program covering the design and control of quality concrete; construction of shell roof projects; design and construction of load bearing concrete masonry walls, casting and erecting curtain wall panels; industrial floor construction; Corps of Engineers pavement construction; and Spancrete, a prestressed concrete floor, roof, and wall unit. Three recently issued Portland Cement Association films were included in the program.

Kenneth D. Cummins, ACI technical director, presented a paper on "Recommendations for General Formwork." ACI members, Norman E. Henning, vice-president, Twin City Testing and Engineering Laboratory, St. Paul, and Ralph E. Spears, regional structural engineer, PCA, Kansas City, were among the speakers on the program.

Toledo CIB hears Cummins

The Toledo area Concrete Improvement Board held its monthly meeting on April 19 with Kenneth D. Cummins, ACI technical director as guest speaker.

Mr. Cummins presented a talk on formwork of structures based primarily on the recent report of ACI Committee 622 "Formwork for Concrete," which appeared in the March, 1961, JOURNAL. His talk was augmented with slides depicting failures due to improper construction of formwork and also a series of slides depicting interesting use of formwork in unique concrete structures including the Lambert Airport Terminal in St. Louis; the Mount Clemens Savings and Loan Association Building, Mount Clemens, Mich.; the TWA hangar at International Airport, New York; and St. Clements Church in Centerline, Mich. Following the presentation an interesting discussion was participated in by James A. Nicholson, Nicholson Concrete Co., Toledo, and others.

MIT schedules special short summer courses in advanced highway engineering

The Massachusetts Institute of Technology will conduct a special summer short course in advanced highway engineering August 14-18.

The course is intended for practicing highway engineers and planners, as well as for those engaged in teaching and research on highway engineering problems. The lectures and discussions in the program will draw heavily on the results of research in highway design now underway at MIT and elsewhere.

Tuition for the course is $175. Academic credit is not offered for the course which will be conducted in full-day sessions.

The program is under the general direction of Alexander J. Bone, professor, Transportation Engineering Division, Department of Civil and Sanitary Engineering.

This is one of 26 such special summer programs scheduled at MIT for 1961.

Scandinavian group to study resistance of concrete in sea water

The Scandinavian Concrete Association has set up a Scandinavian working group on the resistance of concrete in sea water.

At the conference of the Danish Concrete Association in 1959, it was emphasized from various quarters that there was a great need of a common Scandinavian basis for a simple and clear code for concrete technicians for making concrete structures sufficiently resistant to sea water. It was recommended that a Scandinavian working group be set up to deal with the subject.

Poul Skovgaard, civil engineer, Cementfabrikkernes Tekniske Oplysningskontor, Copenhagen, is acting secretary of the working group made up of representatives from Denmark, Finland, Norway, and Sweden.

The group held its first full meeting in September, 1960. During the first year the group will concentrate on a detailed survey and description of concrete structures in sea-water and conditions prevailing on the coasts of the Scandinavian countries. They will work out a provisional code for concrete construction in sea-water.

Foster appoints Texas representative

Christopher J. Foster, consulting engineers of New York City, will be represented in Port Neches, Tex., by Theron P. Davenport, who recently took early retirement from Texaco's Marine Department, after 32 years of service. Mr. Davenport will continue to specialize in terminal and transportation consultation in the petroleum industry and in marina and survey work in the small craft field.

64th annual ASTM meeting to be held June 25-30

The 64th annual meeting of the American Society for Testing Materials will be held at Chalfonte-Haddon Hall, Atlantic City, N.J., June 25-30. More than 3000 engineers and scientists are expected to attend the 5-day meeting which will include technical sessions on some 30 subjects.

For the construction industry a symposium on soil dynamics, and sessions on soils, concrete, and road and paving materials are on the schedule. Nine papers will be presented at the session on concrete. Papers to be presented Monday evening, June 26, will cover the indirect tension test for concrete, correlation of flexural and compressive strengths of concretes and mortars, strain distribution in compressively loaded concrete specimens, and methods for determining mechanical resonance frequencies and for calculating elastic moduli from these frequencies. The session will continue on Tuesday evening, June 27, with papers dealing with application of epoxy sealants for repairing structural cracks in concrete members, variation on pozzolanic behavior of fly ashes, concrete containing fly ash as a replacement for portland blast-furnace slag cement, testing uniformity of large batches of concrete, and modern concepts in control of concrete.

The sessions are open to the public with all interested individuals invited to attend.

Shawalter joins Permanente staff

A. J. Showalter, formerly chief engineer of the San Gabriel Ready Mixed Co., Pasadena, Calif., has been named technical consultant at Seattle for the Permanente Cement Co., Cupertino, Calif. He will supply technical assistance on cement and concrete problems to Permanente customers throughout Oregon, Washington, and Alaska.

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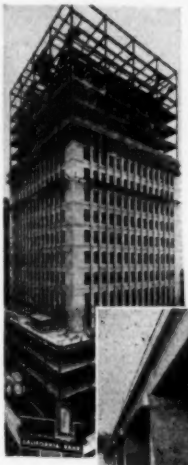

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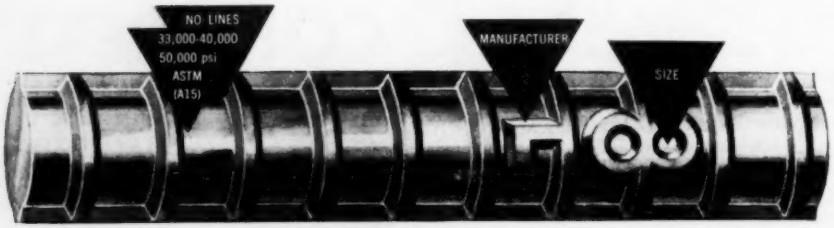
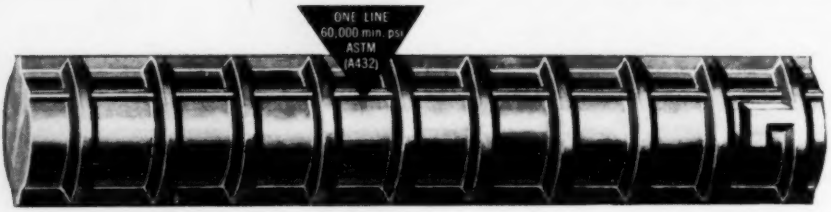
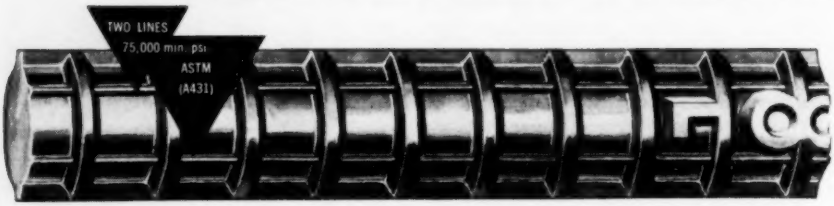


ROLLED-IN MARKINGS



SHOW SIZE AND STRENGTH



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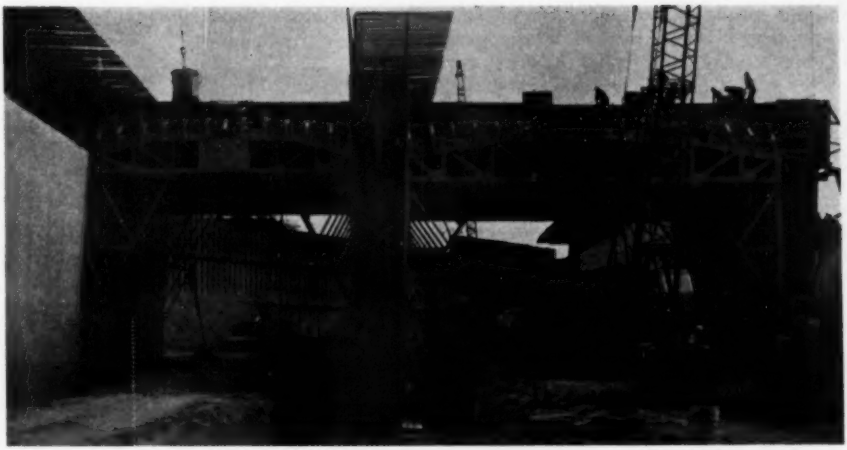

Rigid Frame Bridge Built with Traveling Forms

High-speed concrete placement in a confined working area was simplified by Robert R. Anderson Co., Chicago, through use of specially designed steel traveling forms to construct a three-level grade separation structure in Cook County, Ill. The $5¼ million project is part of the new Northwest Expressway being built under supervision of the Cook County Department of Highways.

The structure will carry Chicago and North Western Railway tracks over the expressway and under an elevated section of Milwaukee Ave. To handle sharp angles of intersection at the grade separation, the structure was designed to include three staggered concrete tubes, each of which is 400 ft long.

A self-supporting curved embankment—2000 ft long and 65 ft wide—was devised to bypass the railroad around the limited working site without obstructing it, and to permit use of the traveling steel forms, designed and fabricated by Blaw-Knox Co.

Span of the two outer tubes is 50 ft, including a 31-ft center span, while clearance is 33½ ft. Concrete walls are 3½ ft thick at base and 5½ ft at deck slabs, which support parabolic arches 7½ ft thick at the walls and 4 ft thick at center span.

Use of the steel forms made possible: (1) monolithic slab placement over intermediate walls; (2) placement of slabs with a load of 750 lb per sq ft; and (3) an open job site for unrestricted movement of materials and equipment during concrete placement.

Each of the 64-ft long Blaw-Knox forms included 14 trusses supported on columns and knee braces, and moved on 8 x 15-in. rubber-tired wheels on a concrete bridge footing. Metal plate, 3/16 in. thick and supported by 6 in. wide junior beams spanning the truss-

es, was employed to place the concrete arch. Screw jacks, housed in the base of the forms, were used to raise and lower forms for traveling.

A total of 36,675 cu yd of concrete were employed to complete the bridge structure. The two largest placements required 1200 cu yd of concrete and simultaneous use of all three sets of traveling steel forms. All concrete cured for 10 days before forms were stripped and repositioned for additional concrete placement.

Although there was not heavy reuse of each set of the forms, the contractor reported they easily amortized their cost through increased speed of concrete placement, quality of finished concrete, and maintenance of job traffic in a comparatively small working area.

Higginson lectures at University of Mexico

Elmo C. Higginson, chief of the Concrete Laboratory Branch, U.S. Bureau of Reclamation, recently completed a 2-week engagement as guest lecturer at the University of Mexico's Graduate School of Engineering in Mexico City. He delivered a series of 11 lectures, discussing the field control of concrete construction and concrete research.

Mr. Higginson, an authority on concrete technology, is the author of numerous technical papers in that field. He has been with the Bureau of Reclamation since 1934, serving in various capacities before his appointment to his present position in March, 1960. He is chairman of ACI Committee 609, Consolidation of Concrete, and a member of Committee 805, Application of Mortar by Pneumatic Pressure.

Civic Administration seeks municipal survey answers from "men-in-the-know"

Civic Administration, Toronto, is looking to consulting engineers, suppliers, and contractors for answers to problems raised time and again at meetings regarding municipal administration of construction contracts.

The publication plans to publish a two-part study in its July and August issues and is relying on the judgment and experience of people directly involved to furnish the answers.

Questionnaires from the consulting engineers', the suppliers', and contractors' points of view are available from *Civic Administration*, 481 University Ave., Toronto, Ont., Canada.

Sika appoints Sieverling to top sales post

Paul A. Sieverling has been appointed sales manager for Sika Chemical Corp., Passaic, N.J.

Mr. Sieverling will be responsible for the company's sales through its 11 district offices and its broad dealer franchises throughout the United States. He brings some 35 years of concrete construction experience to this position.

Errata

The following corrections should be made in "Freezing and Thawing Tests of Lightweight Aggregate Concrete," by Paul Klieger and J. A. Hanson which appeared in the January, 1961, JOURNAL.

p. 784—In Table 4, second column from the right, change "weight" in the heading to "volume."

p. 793—In the abscissa to Fig. 5 change "weight" to "volume."

The following correction should be made in the Concrete Briefs article "Ultimate Strength Design of Beams with the Aid of Tables," by A. Zaslavsky which appeared in the February, 1961, JOURNAL.

p. 980—under the first radical in Eq. (4) the material in brackets should read "$(1 - 0.59\ p\ f_y/f_c')$" instead of "$(1 - 0.59 f_y/f_c')$."

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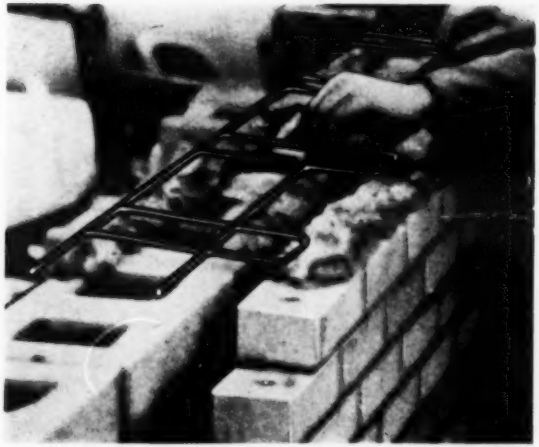

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Wagner assumes new post at Universal Atlas Cement

Fred L. Wagner, formerly vice-president eastern sales region, has been appointed to the newly created position of vice-president customer relations for Universal Atlas Cement, New York.

James E. Taylor, formerly assistant to vice-president general sales, has been appointed as manager of sales eastern region, and will assume the responsibilities formerly held by Mr. Wagner.

Roof shapes spotlighted at BRI spring meeting

New roof shapes, riding a rising tide of interest in the United States over the past 15 years, were the topic for discussion in nine technical reports at the Building Research Institute spring conference, May 16-18, Washington, D.C.

In a conference on requirements for weatherproofing thin shell concrete roofs speakers discussed the initial requirements, the effect of physical factors on weatherproofing, problems and limitations imposed by the roofs themselves, and properties and uses of materials currently available for weatherproofing and sealing thin shell concrete roofs.

Other sessions featured papers on thin shell structures using lift-slab erection process, plastics enclosures for an arch catenary cable suspended roof structure, anisotropic cylindrical shell roofs under external pressure, and tension structures and related research.

Joyce promoted by Hercules

John J. Joyce has assumed the post of New York District sales manager for the Hercules Cement Co., Division of the American Cement Corp., Philadelphia. Mr. Joyce has been with Hercules for 15 years.

Stein forms new consulting firm

J. Stewart Stein, formerly a partner in the firm of Walter H. Sobel—J. Stewart Stein, has formed a new firm for the continuation of the practice of architecture and engineering with offices in Franklin Park, Ill.

Zia joins faculty at N.C. State College

Paul Zia, formerly assistant professor of civil engineering, University of Florida, Gainesville, has joined the faculty of the Department of Civil Engineering, North Carolina State College, Raleigh, N.C.

Executive changes at Penn-Dixie

Fred L. Doolittle has been elected president and chief executive officer of the Penn-Dixie Cement Corp., New York. Mr. Doolittle succeeds B. W. Bruckenmiller, who has been elevated to chairman.

Mr. Doolittle joined Penn-Dixie's Boston sales office in 1929. He has been executive vice-president and director since 1956.

Superior Steel Fabricators named Dewey Almy distributor

The appointment of Superior Steel Fabricators, Inc., Hapeville, Ga., as distributor of the concrete admixtures of the Dewey and Almy Chemical Division, W. R. Grace & Co., Cambridge, Mass., has been announced.

Superior Steel Fabricators will distribute Dewey and Almy's admixtures in Georgia, South Carolina and parts of North Carolina and Tennessee.

ACI member Daniel W. Yike, a concrete technician with close to 30 years experience on mix proportioning and admixtures, has been appointed to the Superior Steel staff as promotion manager for the new product line.

Sika appoints two technical engineers

Sika Chemical Corp., Passaic, N.J., has appointed two engineers to its technical sales and service departments.

F. M. Ackley, technical service engineer, brings 28 years of varied technical, construction and practical experience to his new post. He will serve the concrete construction industry in resolving difficult concrete problems, particularly on shafts, tunnels and large underground structures. He will work out of Sika headquarters in Passaic, New Jersey.

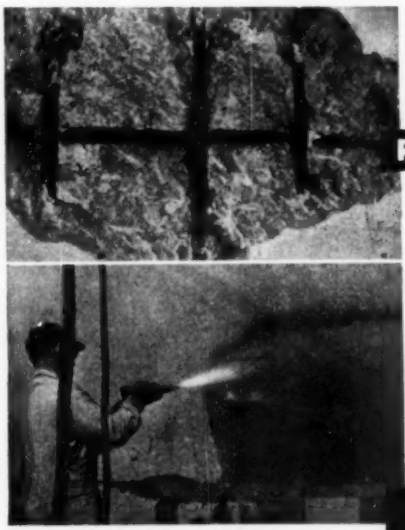
Alex J. Chidichimo, technical sales engineer, will call on architects, engineers, public works officials and industrial firms in the Chicago and northern Indiana areas. A structural engineer, and former partner in the consulting engineering firm of Norman Alexander, Inc., Chicago, he was previously project supervisor for Procon, Inc. of Desplaines, Ill.

CIT announces first alumni conference

Education beyond the diploma will be emphasized in Carnegie Institute of Technology's first Carnegie Conference to be held June 10 on the Carnegie Tech campus.

More than 30 faculty members will participate in 8 symposiums and 12 lectures scheduled for the all day conference. More than 500 alumni are expected to attend.

"Longer Spans — Less Weight," a report on current investigations at Carnegie Tech aimed at development of consistent analysis of stresses and deformations in orthotropic plates with eccentric stiffeners, will be given by ACI member Tung Au, associate professor of civil engineering, CIT.



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by Mel Prior
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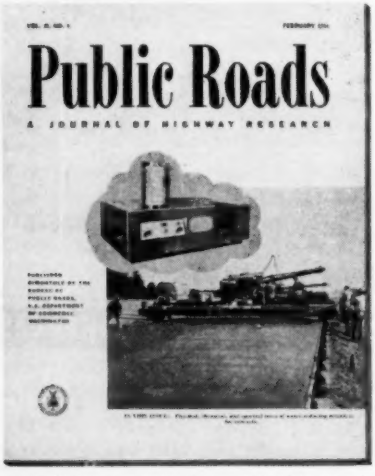
(No. 21 of a series)

Bureau of Public Roads' Water-Reducing Retarder Report

BY COINCIDENCE, this issue of Darex Diary is number 21; same as the code number for Dewey and Almy's DARATARD in the Bureau of Public Roads' recently published study of water-reducing retarders.



The February, 1961 issue of Public Roads was devoted to physical, chemical and spectral tests of water-reducing retarders for concrete.

The members of the Bureau's Division of Physical Research are to be congratulated for planning, organizing, and conducting this series of tests, as well as for an excellent presentation of the test data. The work is a valuable contribution to the knowledge of admixture use in concrete, and an excellent guide for specification writers.

The twenty-five admixtures submitted for testing by the Bureau can be divided into three basic groups: Lignosulfonates, Organic Acids and Carbohydrates. It is interesting to note that these three groups had widely varying effects on the properties of the resultant concrete. Furthermore, individual admixtures within each group had significantly different effects on the properties of the admixtured concrete.

Retardation is the prime consideration in any specification for water-reducing retarders, and should be determined according to ASTM Designation C 403 before investigating other properties. The effect of the retarder on water reduction, strength, durability, and shrinkage of concrete should also be thoroughly investigated. These factors have a bearing on the over-all quality of the admixtured concrete and all should be considered by the specification writer.

In the Bureau's symposium, DARATARD placed easily within the requirements of the recommended specification for water-reducing retarders, and ranked high on the list among all admixtures tested. From a completely impartial standpoint, the Bureau's report points up the wide difference in lignins, with DARATARD being among those showing the most desirable properties. This underscores the important point that the results obtained with any one lignin should not be taken as indicative of the performance of all. The BPR specification offers a good basis for proper evaluation.

Litvin joins PCA research staff

Albert Litvin, formerly supervisor, Masonry Materials Section, Armour Research Foundation of Illinois Institute of Technology, Chicago, has joined the Portland Cement Association research staff in Skokie, Ill.

Mr. Litvin is a member of ACI Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete, and Committee 623, Cellular Concretes.

Bell assumes new post with California L&PCA

Clyde Bell has been appointed managing director of the California Lathing & Plastering Contractors Association.

Wheat assumes new post

T. E. M. Wheat has been named chief structural engineer for the architectural firm of O'Dell, Hewlett and Luckenbach Associates, Birmingham, Mich.

Mr. Wheat was previously head of the structural department at Eberle M. Smith Associates, Detroit architectural and engineering firm and was in charge of the structural design of the Henry Ford Community College, a multi-building campus development currently under construction in Dearborn, Mich.

Sawyer elected chairman of PCA board of directors

H. A. Sawyer, chairman of the board and president of the Lone Star Cement Co., New York, recently was elected chairman of the board of directors of the Portland Cement Association, having served as a board member for 5 years.

Mr. Sawyer joined Lone Star in 1925 as a service engineer, advancing through the ranks until 1952 when he was elected president, a director, and a member of the firm's executive committee. He took on the additional post of chairman of the board in 1959.

Neiman enters private practice

Abraham S. Neiman has retired from the U.S. Air Force and has entered private consulting engineering practice in the Los Angeles area. His office is located in Beverly Hills, Calif.

Sollenberger advanced

Norman J. Sollenberger has been appointed chairman of the civil engineering department at Princeton University to succeed the late William Mack Angas.

Professor Sollenberger joined the Princeton faculty as an instructor in 1941. From 1946 to 1952 he was absent from the university while working as a senior design engineer with John A. Roebling Sons Co., and subsequently as a teacher at Iowa State University. During this absence he also spent 2 years in El Salvador engaged in the construction of the San Marcos suspension bridge.

1962

The Technical Activities Committee has begun its plans for ACI's 58th annual meeting, March 12-15, Denver.

Those who are interested in participating in the program should write to Institute headquarters before July 1, furnishing a synopsis giving the scope of the proposed paper and indicating features that the author thinks will justify its inclusion in the technical program.

Preliminary drafts should reach TAC for approval and acceptance by September 15; final manuscripts will be due by Jan. 1, 1962.

Grading the embankment for construction of stadium seats where spectators will view the "World 600," an annual 600-mile racing event for late model stock cars.

Earth moving equipment speeds construction of concrete stadium

McDevitt & Street Co. simplified construction of a 50,000-seat stadium at the Charlotte Motor Speedway, Charlotte, N. C., with efficient use of earthmoving equipment.

Conventional earthmoving equipment was used to build and compact an embankment with a 2-to-1 slope that served as a base for the grandstand. Then, starting from the top of the embankment, a small 47-hp International T-340 crawler tractor and a 9-hp walk-behind trencher graded for the 25 rows of seats.

First the trencher cut a trench 15 in. deep. Then, the T-340 bladed off 15 in. of earth to the tow of the trench. As the work progressed, the small crawler had an ever larger volume of earth to move as the spoil from the higher rows was added to that of the new cuts to be made below.

Together, the T-340 and walk-behind trencher team did a fast job of grading for 75,000 lineal ft of stadium seats. Reinforced risers were concreted as fast as the two machines graded for them. Then, using an extra-long chute to place concrete for all 26 rows of seats directly, McDevitt & Street cast 4000 lineal ft of 13-in. deep seats per day.

Charlotte Motor Speedway consists of two auto race tracks, an outer 1½-mile oval and a winding 1-mile infield course.

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Vermiculite Institute holds 20th annual meeting

Vermiculite Institute, Chicago, held its 20th annual meeting March 18-23 at Chandler, Ariz.

L. J. Venard of Minneapolis, Minn., was elected to the presidency. R. W. Sterrett of Chicago was named treasurer, and Edward R. Murphy was returned as managing director.

Nicholson retires

Horace B. Nicholson, partner in the firm of Nicholson, Rothbaum and Davis, consulting structural engineers, Philadelphia, has announced his retirement from the firm as of March 1. He will be available for consultation to the firm which will continue its engineering services under the firm name of Rothbaum and Davis.

Louisville Cement announces staff appointments

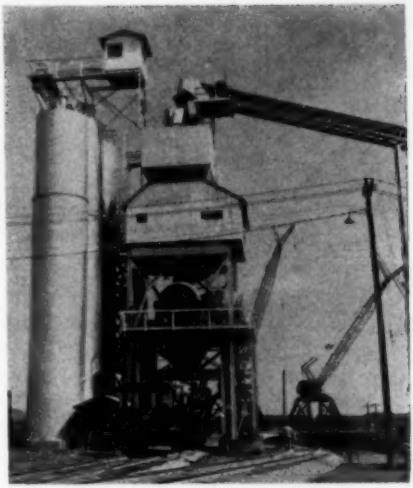
Frank L. Lovell, formerly sales manager, was elected vice-president of the Louisville Cement Co., Louisville, Ky. He joined the company as a salesman in 1940.

Louis B. Hartlage, Jr. has been appointed traffic manager. He has been with the company since 1952.

McFarlan and Skipper address Detroit CIB

Two Atlanta ACI members, J. D. McFarlan, architect, and V. D. Skipper, manager, Ready Mixed Concrete Department, MacDougald Warren, Inc., discussed the work of the Greater Atlanta Concrete Advisory Board before the Concrete Improvement Board of Detroit on March 23.

Following the discussion a panel of Detroit CIB members, representing architects, transit-mix concrete producers, home builders, and general contractors outlined the need for improved performance standards in specific phases of the Detroit concrete construction industry.



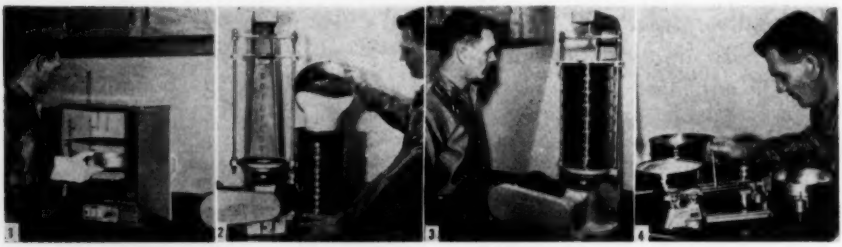
Push-button concrete for Cowans-Ford Dam

Push-button batching and mixing features the 200 cu yd per hr concrete producing operation of Duke Power Co. for construction of Cowans-Ford Dam near Charlotte, N.C. Nine individual weigh batchers and three tilting mixers are operated automatically from a console located on the batching deck. Operator sets up the batch-mix-dump cycles on the console from a selection of six preset formulas of nine different materials. Pre-mixed batches are dumped from the holding hopper into concrete buckets on a flatcar and hauled 600 yd to the placing site. Plant operates 8 hr a day on this 350,000 cu yd job scheduled for completion in early 1963.

Plant has 300 tons of overhead aggregate storage in five compartments charged by a 30-in. belt conveyor, 4500 bbl of cement storage in separate outside silos including 1500 bbl of fly-ash, and three 40-cu yd tilting mixers. Designed on the stack-up principle, the plant was erected in only 2 weeks. All batchers, scale systems, bin gates, air hoses, rams, electrical panel board, controls, and batch deck were sub-assembled at the factory. Plant and batching console were manufactured by Noble Co.

Rosasco named vice-president of Moors Concrete Products

Andrew Rosasco has been named vice-president of Wm. Moors Concrete Products, Inc., Fraser, Mich. He has represented the company in the architectural field for several years.



Sieve Analysis of Fine and Coarse Aggregate—
ASTM C-136 and AASHO T-27

The sieve analysis or gradation of fine and coarse aggregates determines the particle size distribution. Testing sieves of various diameters and size openings are used. From this test, the fineness modulus of the aggregate is determined for relationship of the amount of mix water required in concrete.

Fig. 1—Samples are oven dried to a constant weight at a temperature not exceeding 100 C (230 F). The dried sample is weighed and the weight data recorded. **Fig. 2**—Here 8-in. diameter test sieves are arranged in order with the largest openings on top and a sieve pan at the bottom. The sample is introduced into the top sieve. Care is taken to assure that none of the sample is lost. **Fig. 3**—In the sieve shaker, the sieves are moved laterally and vertically accompanied by a jarring acting to keep the sample moving over the surface of the sieves for a fast gradation process. **Fig. 4**—The weight of material retained on each sieve size and the pan is determined on a scale or balance which is sensitive to within 0.1 percent of the weight of the test sample. Photos courtesy Soiltest, Inc.

Honor Roll

Point System

1 point for Student; 2 points for Junior; 3 points for Individual; 4 points for Corporation; and 5 points for Contributing.

Jan. 1 — Apr. 30, 1961

U. Ersoy holds top position on the Honor Roll for the third consecutive month. Have you signed a new ACI member recently? Among your colleagues working in concrete, there are probably several excellent ACI prospects. Why not tell them about ACI today! Let's keep the membership trend spiraling!

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J. R. Florey	6
J. E. Heer, Jr.	6
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K. K. Nambiar	6
O. O. Pfutzenreuter	6
J. Spinel L.	6
R. G. Swan	6
E. S. Vieser	6
A. Alper	5
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A. R. Flores	5
K. D. Hansen	5
P. W. Abeles	4½
A. W. Brust	4
M. N. Clair	4
W. S. Cottingham	4
T. R. Jones, Jr.	4
J. W. Kelly	4
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M. Nazario Bey	4
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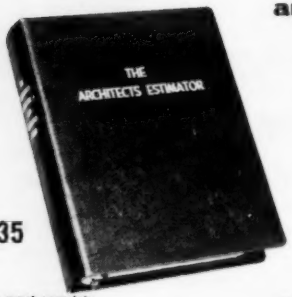

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New Members

The Board of Direction approved applications in the following categories: 74 Individual, 2 Corporation, 17 Junior, and 26 Student, making a total of 119 new members. Considering losses due to deaths, resignations, and nonpayment of dues, the total membership now stands at 10,380.

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BAIRD, EDMOND G., Warren, Pa. (Chf. Draftsman, Pittsburgh Desmoines Steel Co.)

BALTHASER, R. E., Los Angeles, Calif. (Salesman, Master Bldrs. Co.)

BERGERON, MARC, Quebec, Que., Canada (Struct. Engr., Gilles Vandry, Cons. Engr.)

BERNAERT, STEPHANE, Paris, France (Rsch. Engr., Comité Européen du Béton)

BERNUPS, A., Montreal, Que., Canada (Proj. Engr., Alumn. Co. of Canada, Ltd.)

BIELSKI, S. M., Cleveland, Ohio (Constr. Supt., Erie-Lackawanna RR Co.)

BIU, POON YING, Hong Kong, B. C. C. (Struct. Engr., Hong Kong Govt.)

BLUMENSTEAD, STEWART E., Hollywood, Calif. (Chf. Draftsman, Wailes Precast Conc. Corp.)

BOYCE, F. L. Sioux Falls, S. D. (Contr., Sioux Falls Constr. Co.)

BRANDT, CHESTER C., Moline, Ill. (Struct. Engr., Deere & Co.)

BRESNEHAN, RHEBAN WILLIS, Rotorua, New Zealand (Constr. Supt., Clerk of Wks.)

BROWN, RICHARD LEE, Los Angeles, Calif. (Part., Reiss & Brown)

CARLSON, JOSEPH, JR., Shelton, Conn. (Cons. Engr.)

CORBETTA, LOUIS J., New York, N.Y. (Dir., Chf. of Fld. Opers., Corbetta Constr. Co., Inc.)

DAVIE, A. M., Clackmannanshire, Scotland (Consultant)

DAVIES, Ross W., Wellington, New Zealand (Engr., Cubitt Wells, Ltd.)

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ELLIS, SHEPARD A., University City, Mo. (Owner, Shepard A. Ellis, Cons. Engr.)

ESHERICK, JOSEPH, San Francisco, Calif. (Arch.)

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GIVENS, WILLIAM C., Jacksonville, Fla. (Vice Pres., Capitol Prestress Co.)

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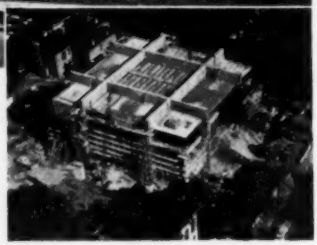

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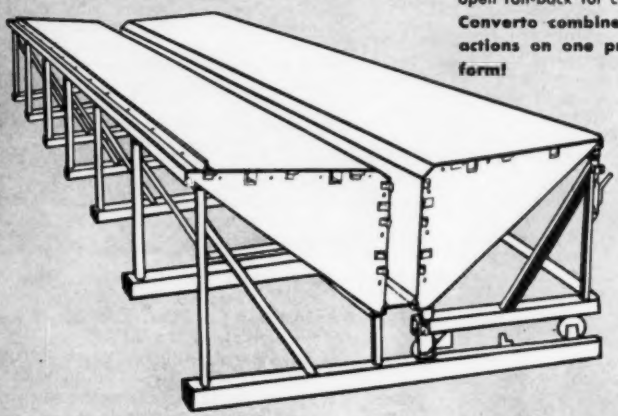

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LOOKING AHEAD

June 12-15, 1961—National Water Resources Research Symposium, Colorado State University, Fort Collins, Colo.

June 25-30, 1961—64th Annual Meeting, American Society for Testing Materials, Chalfonte-Haddon Hall, Atlantic City, N. J.

June 26-July 1, 1961 — 7th International Congress on Large Dams, Rome, Italy

July 3-6, 1961—RILEM Symposium on Durability of Concrete, Prague, Czechoslovakia

July 30-Aug. 13, 1961 — RILEM International Symposium on the Durability of Concrete, Prague, Czechoslovakia

Aug. 11-25, 1961—Prestressed Concrete Conference, Cement and Concrete Association of Australia, Sydney, N. S. W.

Aug. 30-Sept. 2, 1961 — IASS-RILEM Shell Colloquim, Delft, the Netherlands

Sept. 4-6, 1961 — IASS Shell Colloquim, Brussels, Belgium

Oct. 7-10, 1961—Western Building Industries Exposition, Great Western Exhibit Center, Los Angeles

Oct. 16-20, 1961 — Annual Convention, American Society of Civil Engineers, Hotel Statler, New York

Oct. 15-19, 1961—7th Annual Meeting, Prestressed Concrete Institute, Brown Palace and Cosmopolitan Hotels, Denver, Colo.

Oct. 16-17, 1961—Annual Meeting, The Carolinas Ready Mixed Concrete Association, Inc., Jack Tar Poinsett Hotel, Greenville, S. C.

Nov. 1-3, 1961 — 14th Regional Meeting, American Concrete Institute, Dinkler-Tutwiler Hotel, Birmingham, Ala.

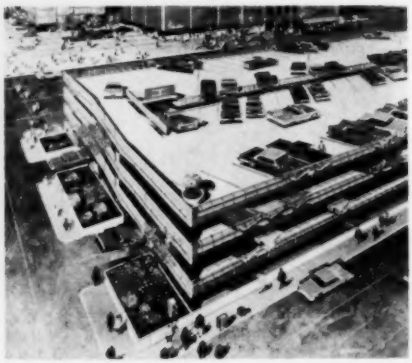
Dec. 5, 1961—First Quality Concrete Conference, University of South Carolina, Columbia, S. C.

Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Precast parking garage

Tishman Research Corp. has developed a new concept in self-parking structures. By standardizing precast and prestressed concrete components in a patent pending process, called Tierpark, Tishman Research has developed a system of mass producing self-



parking structures off-site. The structures are then assembled on site in multiple combinations by a simple, speedy erection procedure.

The height of each tier, floor-to-floor, is 8 ft. Ramp pitch is 12.5 percent. Minimum recommended dimensions are 60 x 147 ft for single tier structures and 116 x 147 ft for multitier structures. Maximum dimensions are unlimited. "Area-per-car" is generally less than 340 sq ft per stall, which is a variable depending on the size and height of the structure.

The primary components of Tierpark consist of: (1) Typical slabs of precast, prestressed concrete, dimensioned for maximum efficiency of structural shape. These typical slabs set the structure's modular dimension and are supported near their center point by single columns. (2) Ramp slabs (down-slabs and up-slabs) which are 1½ times the width of the typical slab and combine in pairs to constitute a complete ramp connecting adjacent parking tiers. They are each supported by two columns. (3) Typical columns which support both typical slabs and ramp slabs. These columns are shaped like elongated hexagons and are tapered at both ends.

The typical slabs, the ramp slabs, and the typical columns form the entire structural system. No beams or girders are required. The rigidly designed connections between columns and slabs eliminate the need for shear walls and bracing to resist wind or seismic forces.

The secondary components consist of: (1) Side railings which are prefabricated and welded in modular units and shaped to provide clearance for bumper overhang and to insure maximum safety and protection. They are easily fastened to the integrally cast curb located at the outer edge of all typical slabs. (2) End railings and curbs which are fabricated as one unit and installed along the ends of each tier of parking and along the ramps. Constructed of metal and concrete, they afford physical and visual protection. (3) Drainage provisions which are built-in to prevent the accumulation of surface water on all tiers. Each typical slab is slightly concave toward the center and weep holes are located at two sides of the column base leading to an internal drain running the full length of each column.

Because of its modular design, the system can be readily adapted to most sites that are suitable for parking and offers unlimited combinations of size and shape that can be developed to satisfy the specific requirements of individual sites. Because of the nature of its components, the system may be designed to allow for future additions. This structure is said to be readily demountable with minimum cost and time loss and can be reassembled at a new location. Designed primarily for structures above ground, the structure can also be adapted for underground parking.

Tishman Research Corp. has licensed 75 prestressed concrete plants across the country to fabricate Tierpark components. In addition, it has retained as quality control consultant, The Freyssinet Co., Inc. — Tishman Research Corp., 666 Fifth Avenue, New York, N.Y.

Liquid plaster

Epo liquid plaster is a pure epoxy resin compound formulated specifically for vertical application to almost all masonry structures. It anchors tight, will not crack on curing, and it said to afford a permanent coating which is practically indestructible.

Manufacturer recommends this product for coating industrial tanks to protect against acids, caustics, alkalies, as well as for coating high and low temperature evaporators. It is also recommended by manufacturer for swimming pools, shower rooms, basement walls, storage pits, silos, and similar applications.

The standard color of Epo liquid plaster is gray, but it is available in a variety of colors. Applied with a trowel, the thickness of coating can vary from over ¼ in. down to a feathered edge. — George E. Fines, Inc., 18640 Wyoming Ave., Detroit 31, Mich.

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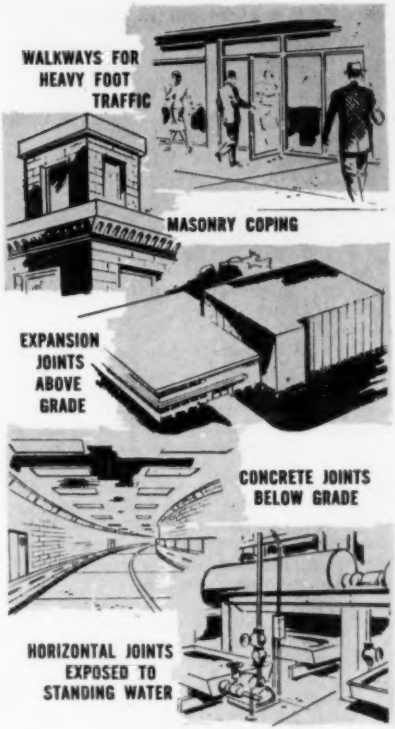
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*Thiokol is a registered name of the Thiokol Chemical Corporation

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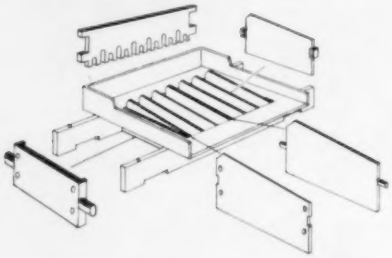
Method of filling concrete block molds

Oswalt Engineering announce the development of the Unifill method of filling concrete block molds. The Unifill mold has been engi-



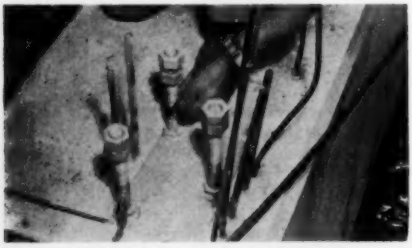
neered to provide the greatest possible uniformity of block and the fastest feeding action for any mold (without the use of agitators and agitator grids) according to company engineers.

As the feed drawer moves forward, gradually tapered dividing plates reach higher up into the feed drawer, shaking the material out in sufficient quantity to assure even filling from the front to the back as the drawer moves over the mold.

Oswalt engineers claim that the cost and down-time involved in a change-over to the Unifill method of mold feedings is small compared to the improvement in the entire mold operation.—Oswalt Engineering Service Corp., 1335 Circle Ave., Forest Park, Ill.

"Glued" anchor bolts

A new technique for positioning anchor bolts for the posts of railings on concrete bridge structures using Sika Epoxy Bonding Compound is said to be faster than the conventional method and produces positive results with estimated savings of 20-30 percent.



This development precludes the necessity of positioning anchor bolts within the formwork prior to placing concrete. It also eliminates tedious and expensive fittings of railing posts, and when properly employed obviates the necessity for shimming—a frequent requirement when anchor bolts are set in the concrete. In addition, the expense of trying anchorages directly to main reinforcement and devising methods of attachment are eliminated.

This technique was developed in the office of the deputy chief engineer (bridges), New York State Department of Public Works. It was used on the Oneida Lake Bridge at Brewerton, N.Y., last summer for the first time, as far as is known.

The concrete fascia beam was cast 4½-in. below final grade with the required tie bars protruding. Three 1 in. round anchor bolts were properly positioned and welded to a 7 x 11 x ½ in. steel plate. Sika Epoxy Bonding Compound was then applied both to a small portion of the concrete beam and to the plate. When the epoxy became tacky, the plate was placed on the epoxy prepared concrete surface. No pressure was required. The plates were positioned with the use of a carpenter's square, measuring from a previously chalked base line on the concrete beam's surface.

Working strength was reached in 8 hr. Specially placed hairpin bars were then bent over the anchorage assembly. At this time the railing post was attached to the anchor bolt assembly. Complete erection of the railing proceeded rapidly. Vertical alignment was accomplished using double nuts on the anchor bolts one above and one below the railing post base. Longitudinal reinforcement was installed and a 4 ½ in. thick concrete cap was placed along the entire fascia beam. — Sika Chemical Corp., 35 Gregory Ave., Passaic, N.J.

All power operated cylinder tester

Forney's announce the Model QC-225-PC compression machine of 400,000 lb capacity. Unlike previous models the unit is completely power operated and the conventional manually operated pump is eliminated. Featuring the new power control unit, the new pump permits rapid traverse for fast preloading and a precise adjustment of the rate of loading. Accessory platens are available for testing masonry units up to and including 12 x 8 x 18 in.—Forney's Inc., Tester Division, P. O. Box 310, New Castle, Pa.

New frontier for burlap

Plain ordinary burlap is progressing beyond the farm and storage barn and making a noticeable impression in concrete construction. Coated with white polyethylene coating, it proves to be highly effective as a curing cover for special shaped concrete designs incorporating in bridge work and building construction.

Burlap when coated with a 4-mil film of white polyethylene is said to be a highly flexible, efficient curing material. Exposed to the atmosphere it reflects solar heat, keeps the concrete at a more consistent temperature, and prevents moisture from escaping. — American Sisalkraft Co., Boston, Mass.

Mortar penetrometer

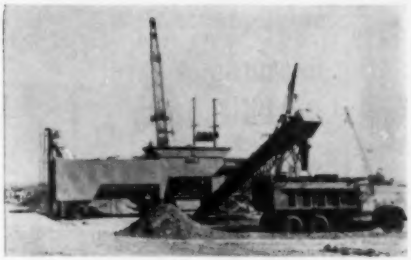
A pocket style concrete mortar penetrometer for field and laboratory evaluation of the initial set of concrete has been announced by Soiltest. The test procedure involves forcing the 1/20 sq in. steel penetrometer shaft into concrete mortar to a depth of 1 in. scribed on the shaft. The resistance in lb per sq in. is shown on the penetrometer's reading scale by the indicator sleeve which automatically holds its position until released. The scale range is from zero to 700 psi.—Soiltest, Inc., 4711 W. North Ave., Chicago 39, Ill.

Portable batching plants

Two portable batching plants that can be relocated in a half day batch concrete at 50 to 60 separate locations in construction of Atlas missile base, Altus, Okla.

Producing dry batch for 25 to 400 cu yd placements, the plants are moved continuously from job site to job site within a radius of 50 to 100 miles. Working around the clock 6 to 7 days a week, each plant produced 50 cu yd per hr of dry batch materials to rigid Corps of Engineers specifications.

Each plant has 40 tons of aggregate storage in three compartments, 1200 cu ft of separate



bulk cement storage, automatic cement and aggregate batching with 3 cu yd batcher, batch-start and batch-dump interlocks, automatic recorder, and 24 in. batch transfer conveyor. Batches are mixed in 34E pavers and placed by concrete pump or buckets into forms.—Nobel Co., 1860 7th St., Oakland, Calif.

Roofing material for shells

A fluid applied roofing material, called Armstrong F/A Roofing, has been developed for use on thin shell concrete roofs of curved angular or undulating slope. The product employs two compounds which are used in varying proportions. The first compound, designated F/A 400, has a Neoprene base

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and serves as a primer and base coat. The second, F/A 600, has a Hypalon base and provides weatherability and color for the surfaces.

According to the manufacturer, applications to include primer base coats, and weather/color coats will be in total recommended thicknesses of 20 dry mils. Application can be by air operated pressure fed rollers, by hand rollers and by conventional spraying equipment. — Armstrong Cork Co. Information Service, Lancaster, Pa.

Small radio units aid post-tensioning job

Two-way miniature personal portable radios (Seiscor Telepath) played an important role in the post-tensioning of a hyperbolic para-

bolodical fieldhouse roof at Okeene, Okla. The fieldhouse roof weighed approximately 450 tons, and was post-tensioned by using four, 100-ton jacks on opposite corners of the roof. Each jack applied 40 tons of pressure to each of four steel rods running horizontally from the dead corners to the corners of the roof being tensioned.

A post-tensioning job of this magnitude required split-second coordination between the jack crews to keep the tensioning rates synchronized. The radio units provided this coordination by permitting instantaneous contact between the crews. — Seiscor Division, Seismograph Service Corp., P.O. Box 1590, Tulsa, Okla.

Premolded concrete joint sealer

Sealfastic, a versatile premolded concrete joint sealer, is said to be thoroughly waterproof and has a 100 percent expansion recovery up to 1 in. This sealer may be used in regular as well as sawn highway joints, for tilt-up, lift-slab and curtain-wall construction, and in swimming pools, and reservoirs.

Sealfastic may be slipped into joint as thin as ⅛ in. sawn, straight or curved. There is said to be no drip or spillage. Sealfastic is reported to last indefinitely, but in case of damage may be replaced quickly with only hand tools. It will not ignite and is fuel resistant.

The sealer consists of 1-in. bats of polyurethane foam, impregnated with a wide choice of types and colors of binders.— National Expansion Joint Co., 1601 Embarcadero, Oakland, Calif.

Film describes shell roof techniques

A motion picture "How to Build New Shapes in Concrete" has been released by the Portland Cement Association. This 16-mm sound-and-color film surveys several of the techniques employed in construction of concrete shell roofs.

The 19-min film describes many of the popular shell forming developments, including movable forms, lightweight scaffolding, precast and earth forms. It also treats the use of prestressing in shell design, as well as the use of shells in lift-slab buildings. Concreting techniques include pneumatically sprayed shells.

"How to Build New Shapes in Concrete" serves as a companion film to the recently released motion picture "New Shapes in Concrete," which deals with the architectural versatility of shell roofs.

Intended for showing to contractors, engineers, architects, and other similarly interested groups, the film is available on loan from the Portland Cement Association's 35 district offices located in major cities of the United States and in Vancouver, British Columbia. — Portland Cement Association, 33 West Grand Ave., Chicago 10, Ill.

Trowel blade control

Kelley has developed a 44 in. power trowel with counterbalanced blade control. The four blades are easily pitch-controlled while they are turning by moving a finger-tip lever on the handle according to the developer.

The apparatus is compact with a low center of gravity for easy operation without gyrating. Heavily constructed for severe usage, the manufacturer recommends the unit for fast, smooth economical floating and finishing of medium-to-large floor areas. — Kelley Machine Division, 285 Chicago St., Buffalo, New York.

Colorful epoxy coating

PlastXcoat, a self-priming epoxy-based coating, has been perfected for a wide variety of applications by Bradco Plastics. The protective coating, available in 20 colors, can be sprayed, rolled, or brushed onto any type of surface under all weather conditions according to company spokesman.

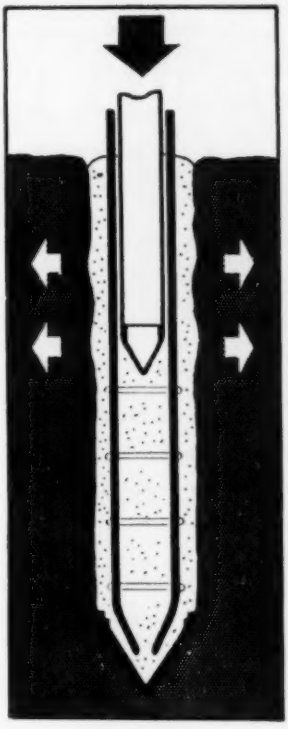
One of the most unusual features about the self-priming coating cited is that it contains a built in plastic "membrane" which eliminates the need for extra coatings. This "membrane" is also said to provide PlastXcoat with superior resistance to the attack of moisture, salt water, corrosive liquids and gases, and prevents chipping, cracking, and peeling.—Bradco Plastics, Inc., 4916 Kirby Drive, Houston, Tex.

Curbing machine

The High Curber, an automatic curbing machine, lays deep curbs without the use of forms and with a minimum of hand finishing according to the manufacturer.

The unit uses a relatively dry or stiff concrete mix at zero slump and lays curbs from 12 to 24 in. high. No forms are necessary as the curb assumes the final form as it is extruded from the machine.

Speed of the machine is said to be 3 to 5 ft per min depending on the size of the curb. Adjustments in the machine allow it to negotiate contours, move up or down grade, and turn radii down to 12 ft. The High Curber, Power Curbers' Model 700, is powered by a Wisconsin Model AGND Engine developing 12.2 hp at 3200 rpm. — Power Curbers, Inc , Salisbury, N. C.



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Power tamper

The Kelley Model 18KT power tamper is especially designed to get in tight places. Manufacturer recommends its use next to foundations and walls, around culverts, piling and piers, in pipeline trenches, and on subbases for road widening jobs.

The tamper is self - propelled, moving itself forward up to 80 ft per minute. It can tamp up to 5400 sq ft per hr. An 18 in. wide shoe is standard on the Model 13KT but a 24 in. shoe is also available. Kelley Machine Division, 285 Chicega St., Buffalo, N. Y.

Self-widening finisher

A self-widening concrete finisher, designed for road and airport paving but versatile enough for many other construction operations including structure paving, has recently been introduced by Blaw-Knox Co.

Operator control is simplified through centrally located electric push buttons and hydraulic operating levels. Also, open grating is used on the floor of the machine to afford good visibility of all paving operations.

The unit is power self-widening through a full range of 12 to 28 ft without frame alteration, in an infinite range of increments, and automatically positions without additional clamping. Wheels are moved from inside to outside hubs for widening from 15 to 31 ft. Various screed arrangements are available in both crown bolt and quick adjustable types. Screed speeds are infinitely variable between 40 to 105 strokes per minute and are independently controlled from traction.—Blaw-Knox Co., Construction Equipment Division, Mattoon, Ill.

Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

A PROGRESSIVE CONCEPT IN THIN SHELL CONSTRUCTION (AIA File 4-A)—This 24-page bulletin describes a technique for construction of thin shell buildings that uses a foamed polystyrene plastic, Styrofoam, as a structural form board to create the desired surface previous to, and act as a support for, the application of the structural roof material and in so doing remain an integral part of the roof as a permanent insulation, vapor barrier, and base for interior decoration once the roof material has attained strength and is a self-supporting structure. The development of Styrofoam is discussed in its use with mortar materials as a plaster base, as a core material for concrete sandwich panels, and as a form liner for the on-site casting of concrete roofs.

A second section discusses wire-supported Styrofoam as a forming technique for walls and roofs. A third section proposes a new concept in thin shell construction. The appendix contains engineering data on the properties of Styrofoam.—Plastics Technical Service, The Dow Chemical Co., Midland, Michigan

CONCRETE PERFORMANCE REPORT: TRI-DAM HYDRAULIC PROGRAM, CALIFORNIA (Bulletin P-75) — Story of the design and placement of more than 500,000 cu yd of concrete for the huge Tri-Dam project in California. Included are studies of the concreting for Tulloch Dam, Beardsley Dam, and Donnells Dam. Mix proportioning, placing techniques, and quality control are covered in some detail. — The Master Builders Co. Cleveland 18, Ohio.

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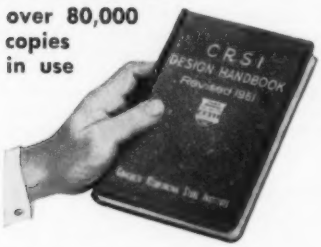

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CONCRETE IN THE '60's: REPORT NO. 1—PCA film, a continuation of the newsreel series issued annually since 1957, highlights outstanding construction projects, news events, and new applications of concrete during the past year. Projects featured make the film suitable for both technical and non-technical audiences. 16-mm prints are available on loan through any of the Portland Cement Association's 35 district offices located in major cities of the United States, including Honolulu, Hawaii, and Vancouver, and British Columbia. — Public Relations Bureau, Portland Cement Association, 33 West Grand Ave., Chicago 10

NEW BEAUTY IN MODERN ARCHITECTURE — Application of Caram Neoprene-Hypalon roofing system to the hyperbolic paraboloidical roof of the Country Club Bowl (bowling alley) in Sacramento, Calif., and the Science and Industry building addition in Los Angeles Exposition Park are described. The elastomer application is performed in three stages. The joints, and breaks around drains, are prepared and sealed with nylon tape and Neoprene coating; a Neoprene spread coating and texturing material is applied; finally a top or color coat of Hypalon is added. — Caram Manufacturing Co., Box 119, Monrovia 45, Calif.

FIRE INSURANCE SAVINGS WITH "FULL FIRE-RESISTIVE" ROOF DECKS — Booklet shows how fire resistive roof decks can often cut fire insurance premiums dramatically (85 percent in one of the examples) and how these savings add up quickly over a few years, pointing out that superior construction can pay for itself through fire insurance savings.—The Flexicore Co., Inc., 1932 East Monument Ave., Dayton 1, Ohio

VIBRO-ENERGY SEPARATOR CASE HISTORIES—Series of publications showing how screening problems in the rock product and other process industries have been solved with the Vibro-Energy Separator. Screening data describes increased production rates, improved separation efficiency, and reduced operating costs. The latest Sweco Separator catalog is also available on request.—Southwestern Engineering Co., Dept. 447, 4800 Santa Fe Ave., Los Angeles 58, Calif.

PIONEER NO. 1008 VAPOR SEAL—Four-page bulletin furnishing information and application data for Pioneer No. 1008 Vapor Seal, a mastic protective coating for corrosion prevention. Surfaces on which it can be used include metals—tanks, bridges, siding and the like; glass; brick; cement; fiber; and a variety of insulators.—Witco Chemical Co., Inc., 122 E. 42nd St., New York 17, N. Y.

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HORN DUROCREX "C" CATALOG—Complete description of Horn's Durocrex "C", a clear, synthetic resin sealer for dustproofing, hardening, and sealing interior concrete floors. Application data, coverage, packaging, time and labor saving characteristics are given.—A. C. Horn Companies, Division, Sun Chemical Corp., 2133 85th St., North Bergen, New Jersey.

TUNNEL CONCRETING PROBLEMS (MBR-P-8)—The successful solution to excavating and concreting problems encountered on 12 major tunneling projects are covered in this 28-page publication, explaining how Pozzolith was used to solve concreting problems. Photographs and accompanying text illustrate the construction techniques of highways, railways, and water tunnels.—The Master Builders Co., Cleveland 18, Ohio

BAUGHMAN CEMENT TRANSPORT (Bulleton A-455)—Literature cites features of the PMCT-1 Cement Transports, including self-unloading by both pneumatic and mechanical means. Complete specifications furnished.—Baughman Manufacturing Co., Jerseyville, Illinois.

DUMPCRETE SIDE-DUMP (Catalog L-140)—Four-page catalog of on-the-job photographs of Dumpcrete side-dump bodies for hauling nonagitated concrete and Dumpcrete spreaders. Diagramatic drawings show important features of the model. Capacity is 6 cu yd; with tandem mounting trailer, 12 cu yd.—Maxon, 2600 Far Hills Ave., Dayton 19, Ohio

FLUME APRONS—An illustrated 2-page report describes design of two 1000 ft long concrete flumes that are subjected to the battering of more than 1000 cords of wood dumped onto them daily. The installation described is the Jesup, Ga., pulp manufacturing plant of Rayonier, Inc. A sketch shows details of construction of the aprons and side walls of the flumes in detail, with dimensions.

Text tells how the flumes were built, with special attention to details of the heavy-duty surfacing containing emery aggregate. The condition of the flumes after 2 and more than 5 years' service, respectively, is also reported.—Walter Maguire Co., Inc., 60 East 42nd St., New York 17, N. Y.

INTERNATIONAL INDUSTRIAL TRACTORS AND EQUIPMENT—1961 catalog illustrating International Harvester Company's line of industrial tractors and equipment. In 20 colorful pages, the catalog covers industrial applications such as digging, earth moving, landscaping, materials handling, mowing, snow removal, cable laying, and pulling big drawbar loads.—Consumer Relations Dept., International Harvester Co., 180 North Michigan Ave., Chicago 1, Ill.

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ACI's world-wide membership is growing in extent and participation —traveling a common road toward better, more economical and durable concrete structures. ACI provides a common ground in the search for and use of new "working tools" in concrete design, manufacture, and erection—and its interpretation.

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MOISTURE CONTENT TESTER (Bulletin C-115-60)—Accurate, fast checking of moisture content of soils and similar materials is described in a 2-page bulletin covering Soiltest's line of Speedy Moisture Testers. Bulletin gives information on the equipment specifications as well as the instructions for use. — Soiltest, Inc., 4711 W. North Ave., Chicago 39, Ill.

CONCRETE PERFORMANCE REPORT: SANDY RIDGE TUNNEL, VIRGINIA (Bulletin P-77) — Describes the longest railroad tunnel built in Virginia and West Virginia, 8270 ft long, which required 61,870 cu yd of concrete. — The Master Builders Co., Cleveland 18, Ohio.

POZZOLITH IN LIGHTWEIGHT CONCRETE (Bulletin MBR-P-14) — The wide use of lightweight aggregate concrete in modern construction is featured in this 20-page publication. Job reports describe the role played by Pozzolith in 13 construction projects. — The Master Builders Co., Cleveland 18, Ohio

CONCRETE DARKENING, DISPERSED BLACK CATALOG — Describes Horn A. E. Dispersed Black, a semi-paste pigment for darkening either air-entrained or regular concrete in monolithic or topping construction. Catalog gives advantages, color values, uses, mixing instructions, testing procedures, and quantity data. — A. C. Horn Companies, Division of Sun Chemical Corp., 2133 85th St., North Bergen, N.J.

EFFLORESCENCE AND CHALKING ON PAINTED MASONRY SURFACES — The causes of efflorescence and early chalking on painted masonry surfaces, and effective methods for overcoming and preventing such deterioration are discussed in this 48-page publication. The booklet defines efflorescence as the deposit of crystalline salts on the surface and in the pores of masonry. Common sources of efflorescent salts are the masonry itself, coats of cement paint, lime, ocean spray, and hard ground waters. These efflorescent salts may attack paints either during application or later in the life of the paint film and cause premature chalking.

Because paint formulation is one of the more important factors in efflorescence, it is given extensive coverage in the publication. The four major types of emulsion vehicles used for exterior masonry paints are compared for resistance to alkali, moisture, heat, and ultra-violet light. Other subjects include: film swelling, film coalescence, paint stability in the presence of soluble salts, pigmentation, pigment volume concentration, pigment dispersion and flocculation, and the effects of titanium dioxide and extenders on chalking. —Gerould Allyn, Resinous Products Division, Rohm & Haas Co., Washington Square, Philadelphia 5, Pa.

EMBECO PREMIXED MORTAR (EPMM-2B) —The use of Embeco Premixed Mortar for achieving tough, strong, nonshrink water-resistant joints and masonry repairs is detailed in this 4-page Master Builders publication. Complete illustrated instructions for use plus comparative compressive strength test results are reviewed.—The Master Builders Co., Cleveland 18, Ohio

CONCRETE PERFORMANCE REPORT: UTICA ART GALLERY (Bulletin P-79) — One of the most imaginative concrete structural systems created in recent years is described. The subject of the bulletin is the Munson-Williams-Proctor Institute Art Gallery in Utica, N.Y., where a gigantic monolithic arrangement of four cast-in-place, post-tensioned concrete girders support the structure's roof and an interior balcony and eliminate all interior columns. The study explains in detailed text and photographs how this impressive structure was erected and how the special concrete was designed, controlled, and placed. — The Master Builders Co., Cleveland 18, Ohio.

Measuring, Mixing & Placing Concrete

RECOMMENDED PRACTICE FOR MEASURING, MIXING AND PLACING CONCRETE (ACI 614-59). **This ACI Standard outlines good practices for measuring and mixing the ingredients for concrete and for placing the finished product. Practices designed to achieve maximum uniformity, homogeneity, and quality of concrete in place are given special consideration. Recommendations cover handling of aggregates, batching procedures, mixing requirements, handling of mixed concrete, methods of placing concrete in forms, and consolidation. 32 pp. $1.00 per copy, 50¢ to ACI members.**

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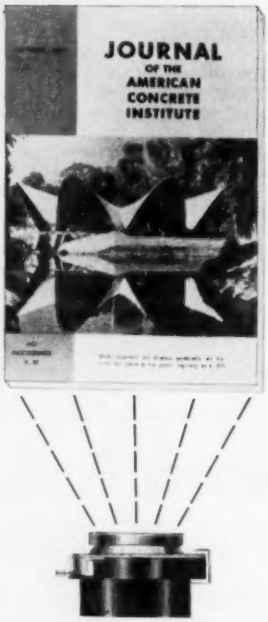

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NOMOGRAPH APPROXIMATES COSTS OF ATOMIC-HARDENED STRUCTURES — A nomograph for computing approximate costs of building atomic-hardened structures has been prepared by Burns and Roe, Inc. The 4-color chart and supporting data were derived empirically from records of actual installations. With the nomograph, rough costs may be estimated for three major types of buildings: rectangular, arch, and dome. Instructions in the use of the nomograph are included, as well as a table describing additional costs per sq ft for architectural, mechanical, and electrical services.—Burns and Roe, Inc., 160 West Broadway, New York 13, New York

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